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DESCRIPTION OF A COMBINED TRIANGULAR AND SUSPENSION BRIDGE TRUSS, AND COMPARISON OF ITS COST WITH THAT OF THE WARREN, PRATT, WHIPPLE AND HOWE TRUSSES.

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GENERAL DESCRIPTION.

The figures, Plate XXX, Class A, from 1 to 15, inclusive, represent side elevations of bridge trusses containing from six to twenty panels. They can be used for iron, steel or combination bridges. (By combination bridge, is meant one in which the top chord, end braces, vertical posts, top lateral struts, and track stringers are wood; all tension members iron; and the intermediate inclined struts and floor beams either wood or iron.) The truss admits of a great variety and choice of figures, but the examples given are, in general, economic and

good selections. The most economic form is, however, governed to a considerable extent by the length of span and the material used.

The figures from 1 to 13, inclusive, are designed for through bridges, but can be used as deck spans, with support at top, by omitting some of the end members, as shown by Fig. 14, which consists of Fig. 9 with parts between A and support omitted, or as deck spans with support at bottom, by adding vertical end posts and extending the top chord, as shown by Fig. 15.

The truss is a combination of the triangular and suspension systems. The primary system is composed of top and bottom chords, and a web of struts and ties arranged in the form of triangles, free to change figure from the effect of temperature. The centre ties extend each over not less than two panels, and over not more than the number in half span less one.

The panel points intermediate between the tops of the centre ties, and not at joints of the primary truss, are supported by vertical suspenders from the top of the centre strut, or by pairs of inclined suspenders directly or indirectly from the tops of the centre strut and centre tie. The inclined suspenders are connected with each other at bottom, but have no fixed connection with the bottom chord, nor with the centre ties at intermediate points of intersection. The top and bottom chords, the centre strut C B, the centre ties A B, and the diagonals A F, F G and G H, or such of them as are found in any figure, constitute the primary truss; all other parts are suspended from it or used to support intermediate panel points. The centre strut may be vertical or inclined, as best fulfills the requirements of any particular case. E is an intermediate panel point supported by the vertical suspender C E from the top of the centre strut C B. D D are intermediate panel points (see detail) supported directly by the inclined suspenders A D and C D from the joints A and C of the primary truss. D^1 in Figs. 9, 11, 12 and 13 is supported by inclined suspenders indirectly from the same points. At I, Fig. 5, the inclined tie is connected with the chord. In detail of suspended panel point D, Plate XXX, the chord is shown supported by a stirrup attached to the pin above. It may, however, be done, if desired, by a seat on the floor beam below, but whatever method is adopted the points D must be left free to move longitudinally unrestrained by the chord. They may be fixed laterally by the floor beam or otherwise, if thought desirable, to prevent vibration.

The weight at any suspended panel point, D or D^1, is conveyed to the joints A and C of the primary truss, and also to the masonry or truss supports, in proportions inversely as its horizontal distance from them. If the part going to C exceeds that which goes to the most distant truss support, its effect is compression on the centre strut and tension on the centre tie on both sides of the centre, and opposed to reverse strains due to unsymmetrical loads at other points. The only panel points on which rolling loads can tend to produce reversed shear in the centre ties are in Figs. 1, 2, 3 and 4, at A; in Fig. 5, at A and I; in Figs. 6 and 7, those between the nearest truss support and A, inclusive; and in Figs. 8, 9, 10, 11, 12 and 13, the same, in addition to the point D or D^1 nearest A, towards the centre. The effect from these latter points is, however, very slight, ranging from one-twenty-eighth to one-tenth of a panel weight; the latter in Fig. 13.

If the centre strut is vertical, as in Figs. 1 and 2, or inclined towards the end, as in Fig. 6, or inclined towards the centre with suspended system extending to support, as in Fig. 14, it can have no tendency to reversed shear. If inclined towards the centre, as in the other figures, it is subject to the same reverse tendency as the centre ties, with the additional effect of panel weight at B.

The compression in the centre strut and tension in the centre tie increases as the points A are moved towards the end of span, and the distance between those points can, if desired, always be made sufficient to prevent any reversed or counter-strain in any member of the truss.

In order to show more clearly the manner in which counter-strains may be avoided, we will take for example Fig. 4, which can be used to advantage for spans ranging from about 130 to 160 feet. For such spans, the rolling load is about two and two-tenths times the dead load. As the inclined suspenders have no fixed connection with the bottom chord, the panel weight at D, nearest A, will be suspended three-fourths from A and one-fourth from C, and the panel weight at D, nearest C, will be suspended one-half from A and one-half from C. The inclined suspenders C D, and vertical suspenders C E, therefore convey to C two and one-half panel weights of dead load, giving a compressive shear of one and one-quarter panel weights on each of the centre struts C B. Adding the panel weight at B, we have a tensile shear of two and one-quarter panel weights on each of the centre ties A B.

The end brace A F takes two and one-quarter panel weights from the

centre tie A B, one and one-quarter panel weights from the inclined suspender A D, and one panel weight from the vertical suspender at A, a total shear of four and one-half panel weights.

We will suppose the rolling load to advance by panel weights from right to left across the bridge. When the load has advanced to A on the right, one-tenth of the panel weight goes to the left support, increasing on the left and relieving on the right by this amount the shear due to dead load on the centre strut and tie; and this is the nearest approach to counter-strain that the centre tie can have, requiring a live load twenty-two and one-half times as great as the dead load to overcome its tension.

As the load advances to the second and third panel points D, the compression on the centre strut and tension on the centre tie are increased, for of the load at those points a greater portion goes to the centre C than to the left truss support. When the load has advanced to the fourth panel point B, it is in position to give the minimum compression or nearest approach to counter-strain that the centre strut can have. In this position of the load, three-fourths panel weight goes to C through the inclined suspenders on the right, and one panel weight goes to the left support. The dead load on the right centre strut is therefore relieved to the extent of one-fourth panel weight of live load, from which it follows that a live load five times as great as the dead load would be required to overcome its compression, the ordinary ratio being, as has been seen, about two and two-tenths. An engine, Mogul, ten-wheel, or consolidation, with or without tender, placed in position for maximum effect, will not relieve the compression on the centre strut by an amount exceeding this. It is seen, therefore, that in this figure there can be no counter-strains.

When the load has advanced to the fifth panel point E, it is in position to give the maximum compression on the centre strut, amounting to a shear of one and one-half panel weights. As the load advances to the sixth, seventh and eighth panel points, the tension on the centre tie is slightly increased, and for a uniform panel load is a maximum at the latter point. In this position of the load, three and six-tenths panel weights go to the left support, and one and one-quarter panel weights go through the inclined suspenders to A on the left. The difference, two and thirty-five-one-hundredths panel weights, is the shear on the tie. A consolidation engine placed with its first driver at the seventh

panel point gives a shear slightly in excess of any other position, but very nearly the same, and not exceeding the above. When the load reaches the ninth and last panel point, we have the maximum shear on the end brace amounting to four and one-half panel weights.

The maximum strain on any web member is found by multiplying its maximum shear by the ratio which the length of the member bears to the depth of the truss.

The maximum strains in the chords are found by adding the horizontal components of the web strains acting upon them when the bridge is fully loaded.

A unit of horizontal strain is a panel weight of dead or rolling load, or both, multiplied by the length of panel and divided by the depth of truss, being the horizontal component of a unit of strain in such web members as extend across one panel only. Each unit of shear in members which extend across two panels exerts two units of horizontal strain, and, in general, the number of units of strain thrown into the chords by any web member equals the number of units of shear in that member, multiplied by the number of panels over which it extends. In the figure under consideration, the bottom chord F B receives four and one-half units of strain from the end brace A F. This is a maximum, for it must be borne in mind that the inclined suspenders have no fixed connection with the chord at points D, and can exert no force upon it. The bottom chord B B receives four and one-half units from chord F B, six and three-quarters units from the centre tie A B, and one and one-quarter units from the centre strut C B, a total of twelve and one-half units of strain. The top chord A C receives four and one-half units from the end brace, one and three-quarters units from the inclined suspenders, and six and three-quarters units from the centre tie, a total of thirteen units of strain.

To the engineer who makes bridges a specialty, the foregoing analysis may appear too minute, but it will enable the general engineer to more readily understand and correctly design the truss. In Table No. 2, following, are given the number of units corresponding to the maximum strain due to dead and live loads in every member of the vertical and lateral systems of Figs. 1 to 14, inclusive.

The usual load specification for railway bridges is two engines of a certain pattern and weight, followed by a train load of a certain weight per lineal foot. To find the maximum strain in every member of any

truss from these wheel loads is, to the occasional calculator, very laborious, and, to the adept, with the advantage of experience and all possible aids, tabular and otherwise, it is not desirable, and the writer is satisfied that, except in rare cases, it is useless labor. A comparison of many cases for various spans, forms of truss and classes of engines, between the method of wheel loads and of the nearest equivalent uniform live load, or such as will produce the same maximum shear on the end brace as the wheel loads, except in a few cases of double intersection, when the length of panel was such as to cause nearly or quite the maximum panel weight from each engine to be borne simultaneously by the same system, gives results so nearly identical as to amount to no essential difference in the distribution of material in the web members, and one method is as likely to satisfy actual conditions as the other. The same is true when applied to the chords, for spans of say 150 feet and under, and is also safe and not impossible to realize in spans of any length.

In a table following are given the uniform live loads which may be substituted for the wheel loads of the leading types of ten-wheel and consolidation engines in spans ranging from 10 to 500 feet. For spans up to 65 feet the loads correspond to a maximum centre moment, and for spans of 70 feet and upwards to a maximum end shear. By the method of uniform panel loads, the maximum strain in any member of the primary truss in any of the figures represented is readily found, and by the method of wheel loads no more labor is involved than when dealing with the Whipple, lattice, sub-trussed Pratt, or Fink suspension trusses. The vertical and inclined suspenders are proportioned for the maximum panel load, or such as results from taking the equivalent tabulated uniform load for the minimum length of span, which will fully load the suspender considered—ordinarily a length of two panels. The bottom lateral system may consist of longitudinal and diagonal tension rods connected with transverse floor-beams, which act as the struts of the system. The longitudinal strains are taken up by the chords at joints of the primary truss, and at intermediate joints in the chord bars, as shown by details of combination bridge, Plate XXXI.

If the floor-beams are connected with the chord bars at each panel point, or if the lateral rods extend across two panels, connecting with the end of floor-beams at chord joints, and with the centre of floor-beam between joints, the longitudinal rods may be omitted.

The bottom lateral system may also consist of transverse struts and

diagonal tension rods connected with longitudinal floor-beams suspended from the panel points, as shown by detail B, Plate XXX.

Defects in Bridge Trusses.

An imperfection may be due to the form of truss or principle on which it acts, which admits of no remedy, or to faults in detail, which may be overcome.

Among the more important defects of common occurrence, the following may be mentioned:

In all combination quadrangular trusses, both main ties and counterties are affected by temperature, the latter frequently to the extent of more than one-half of their ultimate strength. The writer once had occasion to make a calculation for temperature on a 150-foot span combination single intersection truss of ten panels, and found on the counter leading up from the third panel point a strain of 28 000 pounds per square inch, due to a change of 100° in temperature. The amount of strain will, of course, vary with every different span, panel, and distribution of material, but as it may be largely increased by screwing up on the rods, and as the men who look after the bridges on most roads seem to neglect no opportunity of tampering with them, it will be seen that counter-rods are liable to be in a very unhealthy condition, and the quadrangular truss evidently should never be built in combination of materials so dissimilar in character as wood and iron, for the question will naturally arise, if a factor of safety of two is admissible in one member of a truss, why not in all?

The counter-rods in all iron or steel quadrangular trusses, though liable to be overstrained in adjustment by unskilled workmen, are affected by temperature to a much less extent than in combination trusses. The unequal expansion or contraction of different members, due to their unequal size or exposure, will, however, affect them to some extent.

The only trusses free from temperature strains under all conditions are the triangular and suspension systems, or a combination of the two. Many trusses have more or less ambiguity of strain. The double intersection has some, the combined arch and truss has more, and other varieties which might be named a still larger amount. It exists in every truss in which the weight at any panel point can be conveyed to the truss supports by more than one route.

The writer recently had occasion to examine a truss in which it was found that the portion of the weight at the first panel point going to the opposite support had a choice of about 1 600 routes. It is certainly desirable that the strain in every member shall admit of exact calculation, and very essential that doubts be confined to narrow limits.

In quadrangular bridges, the sudden change of strain from counter-tie to the opposite main tie, or the reverse, as a train passes over, and the consequent racking and rattling of parts, is very injurious. When an engine moves at the rate of thirty miles an hour over a span of 150 feet or under, the counters pass from a state of greatest relaxation to one of nearly maximum tension in about 1½ seconds. In the Warren girder the same time will elapse between the maximum direct and inverse strains on any brace, but as there is no lost motion, and the braces toward the centre are proportioned for both direct and counter-strain, and therefore less sensitive to the latter than the lighter members of the quadrangular, the distortion or wave motion of the truss under a passing load will be less manifest.

The Howe truss, which has been used in this country to a greater extent probably than any other, possesses some radical defects, the greatest of which is the insecurity of its bottom chord. An analysis of the strains actually developed in this member is a problem of great complexity. The eccentric manner in which the strain is thrown in and taken up, the tortuous course which it takes in the attempt to equalize itself among the different sticks of which the chord is composed, the defects of framing, the transverse effects of flooring loads, the inequalities of extension due to clamps and consequent twist of angle blocks, precludes the possibility of finding its actual factor of safety. This truss is frequently designed without the aid of an engineer, and when an attempt is made at a calculation, it is necessary, in order to arrive at any results, to start out with suppositions not justified by the facts.

The usual factor of safety covers a multitude of shortcomings of this truss, yet, notwithstanding, the bottom chord frequently fails.

Several years ago, when looking after some bridges on a prominent Southern road, the writer's attention was called to a comparatively new Howe truss with trestle bents under the quarters. His companion, a bridge foreman, well acquainted with the road and its bridges, stated that the bridge was erected about eighteen months previously by an experienced Howe truss bridge contractor. A 64-ton 10-wheel engine

was the heaviest used on the line up to that time. The timber was first-class long-leaf-heart yellow pine, the best bridge timber in the world, and perfectly sound, yet the chord had pulled apart in several places. This bridge, a 200-foot span, had something to do to carry its own weight, hence the result; and it will invariably be found that the accredited factor of safety in the Howe truss is far too high.

In the Warren girder, for braces, on which the maximum strain is tension, but subject also to compression, flat bars connected by zigzag bracing are frequently used. This is objectionable, owing to the tendency of the rivets to shake loose, and to the frequency of damage by handling.

In the combination Warren girder, the usual practice, until quite recently, has been to place the centre of pin at the bottom of the top chord, which is very objectionable. It causes such a severe eccentric strain in that member as to diminish its safety fully one-half. It also involves a change in some of the iron members of the web, in case the wooden parts of the truss are to be replaced by iron. Adjustable counter-ties in quadrangular trusses and trussed ties in triangular trusses are expensive in workmanship.

Both quadrangular and triangular trusses frequently have each end of floor-beam or cross-girder on which the track or flooring rests supported by two suspenders connecting the end of the floor-beam to the joint above. In order to avoid a very large pin at the joint, or excessive fibre strain upon it, these suspenders are frequently placed a considerable distance apart, in the combination Warren, 2 feet or more being not uncommon. The deflection of a floor-beam when the load comes upon it, causes an excess of weight to be thrown upon the inner suspender. This unequal strain is conveyed by the suspender through the pin on which it hangs to the various members of the truss, whose unequal extension or compression gives rise to distortion and cross strains. There is not only danger that one suspender may carry the load intended for both, but the much greater one that incompetent inspectors may screw them up so hard as to cause contrary flexure at the inner one, increasing thereby many times the strain it was designed to take, stretching it far beyond its elastic limit, and, if this treatment is persevered in, ultimately breaking it, though amply strong under proper treatment. For this reason, the writer does not consider adjustments advisable, when used in connection with iron floor-beams. With wooden floor-beams the danger is not

so apparent, as the timber will yield and relieve the suspender to a much greater extent, but in any case, a suspender without screw adjustment is undoubtedly preferable. Detail D, Plate XXX, shows a wooden floor-beam with pin connections. Shrinkage can be provided for by the use of shims.

ADVANTAGES OF THE COMBINED TRUSS.

First.—As there are no confined quadrangles, and as wooden braces with iron tensile connections are not used, there can be no temperature strains.

Second.—As the suspenders act independent of the chord at D, the weight at these points can reach the joints A and C of the primary truss in no other way or manner than as described; and as the primary truss consists of a single system of triangular figures, a weight at any joint of it can go in but one way to either point of support; consequently there can be no ambiguity of strain.

Third.—As the length of any one or more of the members of the primary truss or of the suspenders can be changed, and the work still come together as readily as before, adjustments are uncalled for.

Fourth.—As the centre as well as all other ties are never subject to reversed strain, the effect of dead load being always much more than sufficient to prevent it, zigzag bracing is not necessary.

Fifth.—As the pairs of inclined suspenders at D can be placed as near together as desired, and in contact with the floor-beam suspender between them, the latter, at these points, can be a single bar at the centre of pin without increase of cost. The vertical suspender can be made a single bar, passing through the floor-beam, if the latter is of wood, or in pairs connected by pin with a single loop or plate suspender between them, if an iron floor beam is used. The few remaining panel points, which are located at the joints of the primary truss, may be provided with a single suspender by the use of a larger pin than would be otherwise necessary.

Sixth.—Provision against eccentricity in combination trusses by means of properly constructed chord boxes can be made at small expense, for in the great majority of cases but a single joint in each truss requires such provision.

Seventh.—As there are no adjustments to get out of order, it requires little or no superintendence after it is once erected, and it cannot be injured by unskilled inspectors.

Eighth.—As every part is under strain, whether the rolling load is on or off the bridge, there will be but little distortion or rattling and abrasion of parts, also fewer repairs will be required, and a longer life insured than in counter-braced forms.

Ninth.—It will be found more economical in workmanship than most forms, owing to the absence of sleeve nuts or adjustments of any kind in the truss members, its freedom from zigzag bracing, and its greater uniformity of members. It is also found more economical in material, as shown by the accompanying strains and sections, and wide range of careful comparative estimates of cost.

TABLE No. 1.

NEAREST EQUIVALENT UNIFORM ROLLING LOADS IN POUNDS PER LINEAL FOOT OF BRIDGE, WHICH MAY BE SUBSTITUTED FOR WHEEL LOADS IN THE CALCULATION OF STRAINS WITHOUT MATERIAL ERROR. SPANS FROM 10 TO 500 FEET.

Span, ft.	L. and N. R. R. Cons'n Engine, 83 tons in 57.8 ft., 46.2 tons in 14.8 ft.	N. Y., L. E. and W. R'way Cons'n Engine, 80.7 tons in 54.3 ft., 44 tons in 14.8 ft.	N. O. and N. E. R'way Cons'n Engine, 79 tons in 55 ft., 44 tons in 13.5 ft.	P. C. and St. L. R'way Cons'n Engine, 74 tons in 56 ft., 40 tons in 13.7 ft.	C. S. Railway 10-wheel Engine, 66 tons in 50 ft., 36 tons in 12 ft.
10	4 790	4 800	5 260	5 260	5 540
11	4 360	4 950	5 450	5 450	5 700
12	4 000	5 000	5 500	5 500	5 770
13	4 250	4 960	5 460	5 460	5 730
14	4 400	4 900	5 370	5 370	5 640
15	4 470	4 800	5 280	5 280	5 540
16	4 500	4 680	5 150	5 150	5 400
17	4 480	4 570	5 020	5 020	5 260
18	4 440	4 440	4 880	4 880	5 130
19	4 380	4 420	4 870	4 760	4 980
20	4 320	4 400	4 830	4 610	4 840
22	4 160	4 300	4 720	4 520	4 720
24	4 000	4 160	4 580	4 430	4 670
26	3 940	4 050	4 420	4 380	4 610
28	3 860	3 960	4 260	4 300	4 520
30	3 770	3 860	4 100	4 200	4 420
32	3 670	3 760	4 000	4 090	4 300
34	3 560	3 640	3 920	4 020	4 230
36	3 520	3 540	3 860	3 940	4 150
38	3 460	3 420	3 780	3 880	4 100
40	3 400	3 350	3 710	3 840	4 030
42	3 340	3 300	3 650	3 780	3 960
44	3 260	3 250	3 600	3 720	3 880
46	3 210	3 200	3 540	3 680	3 820
48	3 180	3 150	3 480	3 630	3 746
50	3 140	3 100	3 410	3 590	3 680
55	3 050	3 030	3 340	3 460	3 540
60	2 960	2 950	3 280	3 400	3 430
65	2 870	2 870	3 210	3 340	3 350

Span, ft.	C. S. Railway 10-wheel Engine, 66 tons in 50 ft., 36 tons in 12 ft.	P. C. and St. L. R'way Cons'n Engine, 74 tons in 56 ft., 40 tons in 13.7 ft.	N. O. and N. E. R'way Cons'n Engine, 79 tons in 55 ft., 44 tons in 13.5 ft.	N. Y., L. E. and W. R'way Cons'n Engine, 80.7 tons in 54.3 ft., 44 tons in 14.8 ft.	L. and N. R. R. Cons'n Engine, 83 tons in 57.8 ft., 46.2 tons in 14.8 ft.	Average uses.	2 83-ton L. and N. Cons'n Engines, followed by 2 240 lbs. per lin. ft. freight load.	Number of panels assumed.
70	2 920	2 920	3 210	3 270			3 270	6
75	2 920	2 930	3 210	3 220			3 220	6
80	2 920	2 940	3 220	3 210			3 180	6
85	2 920	2 950	3 220	3 290			3 250	6
90	2 860	2 900	3 170	3 250			3 220	6
95	2 860	2 900	3 160	3 250			3 210	6
100	2 860	2 880	3 160	3 210			3 180	6
110	2 840	2 860	3 120	3 180			3 140	8
120	2 790	2 840	3 080	3 170			3 140	8
130	2 770	2 810	3 040	3 120			3 070	10
140	2 710	2 780	3 010	3 110			3 070	10
150	2 670	2 750	2 970	3 070			3 060	10
160	2 630	2 730	2 930	3 030			3 020	10
170	2 600	2 700	2 900	2 990			2 990	12
180	2 580	2 670	2 870	2 980			2 960	12
190	2 540	2 640	2 840	2 960			2 980	12
200	2 530	2 610	2 800	2 910			2 910	12
225	2 440	2 560	2 720	2 850			2 850	14
250	2 390	2 530	2 670	2 810			2 820	14
275	2 360	2 490	2 630	2 770			2 770	14
300	2 340	2 440	2 570	2 710			2 720	14
325	2 320	2 420	2 530	2 700			2 690	16
350	2 240	2 390	2 500	2 660			2 650	16
375	2 220	2 360	2 460	2 620			2 620	16
400	2 200	2 350	2 430	2 610			2 620	18
425	2 180	2 330	2 420	2 590			2 600	18
450	2 160	2 300	2 390	2 570			2 570	18
475	2 140	2 290	2 380	2 560			2 550	20
500	2 120	2 280	2 360	2 540			2 540	20

TABLE No. 2.

Panel Weight Multiples Corresponding to Maximum Strains in the Vertical and Lateral Systems of Bridges, Class A, Plate XXX.

Fig. 1. 6 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	2.5	2.5	k/d	
Bd	−	1.0	1.167	"	
Dd	+	1.5	1.0	"	
Bc = Dc	−	0.5	0.5	"	2
Bb	−	0.5	1.0	....	2
Cc	+	0.5	0.0		
Chords.					
ad	−	2.5	2.5	l/d	
BD	+	5.0	5.0	"	
Top Lat.					
BC	−	1.5		k/b	
C	+ or −	1.0			
Bot. Lat.					
ab	−	2.5	2.5	k/b	
bc	−	1.5	1.167	"	
cd	−	0.5	1.0	"	
Bot. Long'l.					
ab	−	2.0	2.333	l/b	
bc	−	2.5	2.5	"	
cd	−	4.0	4.0	"	

Fig. 2. 8 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	3.5	3.5	k/d	
Be	−	1.5	1.625	"	
Ee	+	2.5	2.0		
Bc=Ed	−	0.667	0.667	"	2
Bd=Ec	−	0.333	0.333	"	2
Bb	−	0.5	1.0	....	2
Cc=Dd	+	0.5	0.0		

Fig. 2. 8 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Chords.					
ac=ce	−	3.5	3.5	l/d	
BE	+	9.333	9.333	"	
Top Lat.					
BC	− or +	2.5		k/b	
C	+ or −	2.0			
Bot. Lat.					
ab	−	3.5	3.5	k/b	
bc	−	2.5	2.625	"	
cd	−	1.5	1.875	"	
de	−	0.5	1.25	"	
Bot. Long'l.					
ab	−	2.5	2.625	l/b	
bc	−	3.5	3.5	"	
cd	−	0.5	1.25	"	
de	−	1.5	1.875	"	

Fig. 3. 8 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	3.5	3.5	k/d	
Bd	−	1.833	1.959	"	
Ed	− or +	0.833	0.417 / 1.25	"	
Bc	−	0.667	0.667	"	2
Ec	−	0.333	0.333	"	2
b=EeB	−	0.5	1.0	....	2
Cc=Dd	+	0.5	0.0		
Chords.					
ad	−	3.5	3.5	l/d	
dd	−	8.0	8.0	"	
BE	+	7.833	7.833	"	
Top Lat.		See	Fig 2.		
Bot. "		"	"		

TABLE No. 2—*Continued.*

Fig. 3. 8 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Bot. Long'l.					
ab	−	4.0	4.25	l/b	
bc	−	3.5	3.5	"	
cd	−	6.0	6.0	"	
de	−	0.0	1.25	"	

Fig. 4. 10 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	4.5	4 5	k/d	
Be	−	2.25	2.35	"	
Fe	− +		0.25 1.50	"	
Bc	−	1.25 0.75	0.75	"	2
Bd=Fd	−	0.50	0.50	"	2
Fc	−	0.25	0.25	"	2
Bb=Ff	−	1.0	0.5		2
Cc=Dd &c.	+	0.0	0.5		
Chords.					
ac=ce	−	4.5	4.5	l/d	
ce	−	12.5	12.5	"	
BF	+	13.0	13.0	"	
Top Lat.					
BC	− +	3.5		k/b	
C	+ −	3.0			
Bot. Lat.					
ab	−	4.5	4.5	k/b	
bc	−	3.5	3.6	"	
cd	−	2.5	2.8	"	
de	−	1.5	2.1	"	
ef	−	0.5	1.5	"	
Bot. Long'l.					
ab	−	3.5	3.6	l/b	
bc	−	4.5	4.5	"	
cd	−	1.5	2.1	"	
de	−	2.5	2.8	"	
ef	−	0.0	1.5	"	

Fig. 5. 10 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
ab	+	4.5	4.5	k/d	
Be	−	2.0	2.3	"	
Fe	− +		0.5 1.5	"	
Bc	−	1.0	1.0	"	2
Bd=Fd	−	0.5	0.5	"	2
Bb=Ff	−	0.5	1.0		2
Cc=Dd &c.	+	0.5	0.0		
Chords.					
ac	−	4.5	4.5	l/d	
ce	−	5.5	5.5	"	
ee	−	12.5	12.5	"	
BF	+	12.5	12.5	"	
Top Lat.	..	See	Fig. 4.		
Bot. "	..	"	"		
Bot. Long'l.	..	"	"		

Fig. 6. 12 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	5.5	5.5	k/d	
Bc	−	4.5	4.583	"	
cD	+	3.5	3.75	"	
Dg	+ −		0.50 2.50	"	
De=Fe	−	2.0 0.5	0.5	"	2
Bb=Dd, &c.	−	0.5	1.0		2
Fg	+	1.5	1.5	"	3
Cc=Ee, &c.	+	0.5	0.0		
Chords.					
ac	−	5.5	5.5	l/d	
ce=eg	−	13.5	13.5	"	
BD	+	10.0	10.0	"	
DF	+	20.0	20.0	"	
FF	+	18.5	18.5	"	
Top Lat.					
BC	− +	4.5		k/b	
C	+ −	4.0			
CD	− +	3.5		"	
D	+ −	3.0			

TABLE No. 2—*Continued.*

Fig. 6. 12 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Bot. Lat.					
ab	−	5.5	5.5	k/b	
bc	−	4.5	4.583	"	
cd	−	3.5	3.75	"	
de	−	2.5	3.0	"	
ef	−	1.5	2.333	"	
fg	−	0.5	1.75	"	
Bot. Long'l.					
ab	−	4.5	4.583	l/b	
bc	−	5.5	5.5	"	
cd	−	2.5	3.0	"	
de	−	3.5	3.75	"	
ef	−	0.5	1.75	"	
fg	−	1.5	2.333	"	

Fig. 7. 12 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	5.5	5.5	k/d	
Bc	−	4.5	4.583	"	
Cc	+	4.0	3.75		
Cf	−	2.25	2.5		
Gf	−	.....	0.50	"	
	+	1.25	1.75	"	
Cd	−	0.75	0.75	"	2
Ce=Ge	−	0.5	0.5	"	2
Gd	−	0.25	0.25	"	2
Bb=Gg	−	0.5	1.0		2
Dd=Ee, &c.	+	0.5	0.0		
Chords.					
ac	−	5.5	5.5	l/d	
cf	−	10.5	10.0	"	
ff	−	18.0	18.0	"	
BC	+	10.0	10.0	"	
CG	+	18.5	18.5	"	
Top Lat.	..	See Fig. 6.			
Bot. Lat.	..	"	"		
Bot. Long'l					
ab	−	4.5	4.583	l/b	
bc	−	5.5	5.5	"	
cd	−	4.0	5.0	"	
de	−	3.5	3.75	"	
ef	−	6.0	6.5	"	
fg	−	0.0	1.75	"	

Fig. 8. 14 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	6.5	6.5	l/d	
Bc	−	5.5	5.571	"	
cD	+	4.5	4.714	"	
Dg	−	2.25	2.714	"	
Hg	−	.....	0.75		
	+	1.25	2.00	"	
De	−	0.75	0.75	"	2
Df=Hf	−	0.50	0.50	"	2
He	−	0.25	0.25	"	2
Bb=Dd=Hh	−	0.5	1.0	..	2
Cc=Ee, &c.	+	0.5	0.00		
Chords.					
ac	−	6.5	6.5	l/d	
ce=eg	−	16.5	16.5	"	
gg	−	24.5	24.5	"	
BD	+	12.0	12.0	"	
DH	+	25.0	25.0	"	
Top Lat.					
BC	−	5.5	.....	k/b	
	+				
C	+	5.0			
	−				
CD	−	4.5	.....	"	
	+				
D	+	4.0			
	−				
Bot. Lat.					
ab	−	6.5	6.5	k/b	
bc	−	5.5	5.571	"	
cd	−	4.5	4.714	"	
de	−	3.5	3.929	"	
ef	−	2.5	3.214	"	
fg	−	1.5	2.571	"	
gh	−	0.5	2.00	"	
Bot. Long'l.					
ab	−	5.5	5.571	l/b	
bc	−	6.5	6.5	"	
cd	−	3.5	3.929	"	
de	−	4.5	4.714	"	
ef	−	1.5	2.571	"	
fg	−	2.5	3.214	"	
gh	−	0.0	2.0	"	

TABLE No. 2—*Continued.*

Fig. 9. 14 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Panel Weight Multiple. Rolling Load.	R	N
Web.					
aB	+	6.5	6.5	$\frac{k}{d}$	
Bc	−	5.5	5.571	"	
cD	+	4.5	4.714	"	
Dg	−	2.125	2.714	"	
Hg	−	....	0.75	"	
	+	1.625	2.00	"	
De'	−	1.375	1.25	"	3
Hg'	−	1.125	0.75	"	3
e'f=g'f	−	0.875	0.75	"	3
Ff	+	1.25	0.5	....	2
Fe'	−	0.5	0.5	"	2
Bb=Dd=Hh	−	0.5	1.0	....	2
e'e	−	0.5	1.0	....	2
Fg'	−	0.25	0.0	"	
Cc	+	0.5	0.0		
Ee'=Gg'	+	0.5	0.0		
Chords.					
ac	−	6.5	6.5	$\frac{l}{d}$	
ce=eg	−	16.5	16.5	"	
gg	−	24.5	24.5	"	
BD	+	12.0	12.0	"	
DF	+	25.62	25.75	"	
FH	+	25.12	24.75	"	
Top Lat.	..	See	Fig. 8.		
Bot. "	..	"	"		
Bot. Long'l.	..	"	"		

Fig. 10. 16 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Panel Weight Multiple. Rolling Load.	R	N
Web.					
aB	+	7.5	7.5	$\frac{k}{d}$	
Bd	−	5.833	5.895	"	
Ed	+	4.833	4.395	"	
Eh	+	.....	0.687		
	−	2.25	2.937	"	
Ih	−	.....	1.0		
	+	1.25	2.25	"	
Bc	−	0.667	0.667	"	2
Ec	−	0.333	0.333	"	2
Ef	−	0.75	0.75	"	2
Eg=Ig	−	0.50	0.50	"	2
If	−	0.25	0.25	"	2
Bb=Ee=Ii	−	0.5	1.0	....	2
Cc=Dd, &c.	+	0.5	0.0		

Fig. 10. 16 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Panel Weight Multiple. Rolling Load.	R	N
Chords.					
ad	−	7.5	7.5	$\frac{l}{d}$	
df=fh	−	24.0	24.0	"	
hh	−	32.0	32.0	"	
BE	+	19.833	19.833	"	
EI	+	32.5	32.5	"	
Top Lat.					
BC	−	6.5	.....	$\frac{k}{b}$	
C	+	6.0			
CD	−	5.5		"	
D	+	5.0			
Bot. Lat.					
ab	−	7.5	7.5	$\frac{k}{b}$	
bc	−	6.5	6.562	"	
cd	−	5.5	5.687	"	
de	−	4.5	4.875	"	
ef	−	3.5	4.125	"	
fg	−	2.5	3.437	"	
gh	−	1.5	2.812	"	
hi	−	0.5	2.25	"	
Bot. Long'l.					
ab	−	12.0	12.125	$\frac{l}{b}$	
bc	−	7.5	7.5	"	
cd	−	14.0	14.0	"	
de	−	3.5	4.125	"	
ef	−	4.5	4.875	"	
fg	−	1.5	2.812	"	
gh	−	2.5	3.437	"	
hi	−	0.0	2.25	"	

Fig. 11. 16 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Panel Weight Multiple. Rolling Load.	R	N
Web.					
aB	+	7.5	7.5	$\frac{k}{d}$	
Bd	−	5.833	5.895	"	
Ed	+	4.833	4.895	"	
Eh	+	.....	0.687		
	−	2.125	2.937	"	
Ih	−	.....	1.0		
	+	1.625	2.25	"	
Ef'	−	1.375	1.25	"	3
Ih'	−	1.125	0.75	"	3
f'g=h'g	−	0.875	0.75	"	3

TABLE No. 2—*Continued.*

Fig. 11. 16 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
Gg	+	1.25	0.5	k	
Gf'	−	0.5	0.5	d	2
Bc	−	0.667	0.667	"	2
Ee	−	0.333	0.333	"	2
Bb=Ke=Ii	−	0.5	1.0	....	2
f'f	−	0.5	1.0	....	2
Gh'	−	0.25	0.0	"	
Cc=Dd	+	0.5	0.0		
Ff'=Hh'	+	0.5	0.0		
Chords.					
ad	−	7.5	7.5	l	
df=fh	−	24.0	24.0	d	
hh	−	32.0	32.0	"	
BE	+	19.833	19.833	"	
EG	+	33.125	33.250	"	
GI	+	32.625	32.25	"	
Top Lat.	....	See	Fig. 10.		
Bot. Lat.	....	"	"		
Bot. Long'l.	....	"	"		

Fig. 12. 18 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	8.5	8.5	k	
Bc	−	6.125	6.305	d	
Fe	+	5.625	5.305	"	
Fi	+	.....	0.917	"	
	−	2.125	3.167		
Ki	−	.....	1.25		
	+	1.625	2.50	"	
Bc'=Fg'	−	1.375	1.25	"	3
Fe'=Ki'	−	1.125	0.75	"	3
c'd=e'd	−	0.875	0.75	"	3
g'h=i'h	−	0.875	0.75	"	3
Dd=Hh	+	1.25	0.5	....	2
Dc'=Hg'	−	0.5	0.5	"	2
Bb=Ff=Kk	−	0.5	1.0	....	2
c'c=g'g	−	0.5	1.0	....	2
De'=Hi'	−	0.25	0.0	"	
Cc'=Ee',&c	+	0.5	0.0		
Chords.					
ac=ce	−	8.5	8.5	l	
eg=gi	−	32.5	32.5	d	
ii	−	40.5	40.5	"	
BD	+	29.625	29.75	"	
DF	+	29.125	28.75	"	
FH	+	41.625	41.75	"	
HK	+	41.125	40.75	"	

Fig. 12. 18 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Top Lat.					
BC	−	7.5	.....	k	
C	+	7.0	.....	b	
CD	−	6.5	.....	"	
D	+	6.0	.....		
Bot. Lat.					
ab	−	8.5	8.5	k	
bc	−	7.5	7.555	b	
cd	−	6.5	6.667	"	
de	−	5.5	5.833	"	
ef	−	4.5	5.055	"	
fg	−	3.5	4.333	"	
gh	−	2.5	3.667	"	
hi	−	1.5	3.055	"	
ik	−	0.5	2.50	"	
Bot. Long'l.					
ab	−	7.5	7.555	l	
bc	−	8.5	8.5	b	
cd	−	6.5	5.833	"	
de	−	6.5	6.667	"	
ef	−	3.5	4.333	"	
fg	−	4.5	5.055	"	
gh	−	1.5	3.055	"	
hi	−	2.5	3.667	"	
ik	−	0.0	2.50	"	

Fig. 13. 20 Panels—Through.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
aB	+	9.5	9.5	k	
BC	+	9.0	9.0	d	
Cf	−	6.125	6.4	"	
Gf	+	5.625	5.4	"	
Gk	+	.....	1.15	"	
	−	2.125	3.4		
Lk	−	.....	1.50	"	
	+	1.625	2.75		
Cd'=Gh'	−	1.375	1.25	"	3
Gf'=Lk'	−	1.125	0.75	"	3
d'e=f'e	−	0.875	0.75	"	3
h'i=k'i	−	"	"	"	3
Cc	−	1.0	1.5	....	3
Ee=Ii	+	1.25	0.5	....	2

TABLE No. 2—*Continued.*

Fig. 13. 20 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load	Rolling Load.	R	N
Web.					
Bc.....	+	0.5	0.5	k/d	2
Ed'=Ih'...	−	0.5	0.5	"	2
Bb=d'd=h'h	−	0.5	1.0	....	2
Gg=Ll.....	−	0.5	1.0	....	2
Ef'=Ik'....	−	0.25	0.0	"	
Dd=Ff'....	+	0.5	0.0		
Chords.					
ac.....	−	19.0	19.0	l/d	
cd=df.....	−	18.0	18.0	"	
fh=hk.....	−	42.0	42.0	"	
kk.....	−	50.0	50.0	"	
CE.....	+	39.125	39.25	"	
EG.....	+	38.625	38.25	"	
GI.....	+	51.125	51.25	"	
IL.....	+	50.625	50.25	"	
Top Lat.					
BC.....	−	8.5	.....	k/b	
C.....	+	8.0			
CD.....	−	7.5	.....	"	
D.....	+	7.0			
Bot. Lat.					
ab.....	−	9.5	9.5	k/b	
bc.....	−	8.5	8.55	"	
cd.....	−	7.5	7.65	"	

Fig. 13. 20 Panels—Con'd.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Bot. Lat.					
de.....	−	6.5	6.8	k/b	
ef.....	−	5.5	6.0	"	
fg.....	−	4.5	5.25	"	
gh.....	−	3.5	4.55	"	
hi.....	−	2.5	3.9	"	
ik.....	−	1.5	3.3	"	
kl.....	−	0.5	2.75	"	

Fig. 14. 8 Panels—Deck.

Member.	+ or −	Panel Weight Multiple. Dead Load.	Rolling Load.	R	N
Web.					
Ad.....	−	2.0	2.0	k/d	
Ed.....	+	2.0	2.0	"	
Ab=Ed....	−	1.5	1.5	"	4
bc=d'c....	−	1.0	1.0	"	4
Cc.....	+	2.0	2.0	....	4
Cb=Cd'....	−	1.5	0.5	"	2
Bb=Dd'...	+	1.0	1.0	....	2
Chords.					
dd.....	−	8.0	8.0	l/d	
AE.....	+	9.0	9.0	"	
Top Lat.					
AB.....	−	3.5	3.5	k/b	
B.....	+ / −	3.0	3.062		

TABLE No. 2.

Notation.

R = ratio or shear multiple.
N = minimum number of panels loaded for maximum strain.
l = length of panel.
k = length of member.
d = depth or vertical component of member.
b = width of lateral system.
W = wind force at top of end brace.
M = bending moment.

The strains in portal bracing cannot well be represented in the table. They are shown on Plate XXX, and will be found much greater than are usually provided for. Although the writer does not understand the necessity of adding largely to the cost of a bridge by providing against tornadoes sufficient to hurl an ordinary train from the track, with a working strain on the material far within the elastic limits, and which would be safe if they were of daily and hourly occurrence, yet, if this extraordinary provision is made, it should evidently be observed in the portals.

Specifications.

The following specifications have been observed in all estimates following:

Dead load. Rolling load. } See table of strains and sections.

Impact.	Inclined suspenders	20 %
	Long vertical suspenders and counters	25 %
	Iron floor-beams and stringers (panel length under 20 ft.)	25 %
	" " " " (" " over 20 ft.)	20 %
	Floor-beam hangers	50 %
Wind.	30 lbs. per square foot of exposed surface considered as a uniform load, and 300 lbs. per lineal foot considered as a rolling load.	
Unit strains tension.	Wrought-iron truss members, 75′ span	9 000 lbs.
	" " " " other spans	10 000 "
	Steel " " " "	14 000 "
	Bot. flange of iron floor-beams and stringers	10 000 "
	Wind bracing, iron	15 000 "
	" " steel	21 000 "
	Timber, yellow pine (Howe truss)	1 200 "

Unit strains compression.

Wrought-iron.

$$\text{Sqr. Sqr.}\quad \frac{8\,000}{1+.004\frac{l^2}{r^2}} \qquad \text{Sqr. Pin.}\quad \frac{8\,000}{1+.006\frac{l^2}{r^2}} \qquad \text{Pin. Pin.}\quad \frac{8\,000}{1+.008\frac{l^2}{r^2}}$$

l = length in feet, and r = radius of gyration in inches.

Steel.

Under 16 diameters, 14 000 lbs.
Over 16 " substitute 11 000 for 8 000 above.

Yellow pine.

$$\text{Sqr. Sqr.}\quad \frac{1\,050}{1+.002\left(\frac{l}{d}\right)^2} \qquad \text{Sqr. Pin.}\quad \frac{1\,050}{1+.003\left(\frac{l}{d}\right)^2} \qquad \text{Pin. Pin.}\quad \frac{1\,050}{1+.004\left(\frac{l}{d}\right)^2}$$

$\frac{l}{d}$ = ratio of length to least dimension.

Top flange of iron floor-beams and stringers (gross). 8 000
Yellow pine floor-beams and stringers 1 000

Alternate tension and compression.

Maximum compression + maximum tension (iron).. 10 000
" " only. See compression formulæ above.
Use the one giving the greatest result.

Shearing.	Iron..... 7 500 lbs.	Steel..... 10 000 lbs.
Bearing.	Pins..... 12 500 "	" ..... 18 000 "
	Rivets ..... 15 000 "	" ..... 20 000 "
Bending.	Pins..... 15 000 "	" ..... 20 000 "

Estimates of Quantities and Cost.

Classification.

The classification of iron in all estimates following is the same as now in force at the Union Iron Mills in Pittsburgh.

Weights.

The weights given are in all cases gross or mill weights. For finished weights a proper allowance must be made for waste in manufacture.

Cost.

The rates for iron are those ruling about a year ago, at which time several of the estimates were made, as they are thought to be none too high as an

average for different times and seasons; all later estimates have been made to conform thereto. They are as follows:

Wrought-iron, single rolled	2.60 cents.
Rod iron	2.75 "
Double rolled	3.00 "
Plates and angles	3.00 "
Beams and rivets	4.00 "
Channels	4.20 "
Nuts	10.00 "
Steel	4.00 "

To the resulting average price the following additions have been made for manufacture, profit and transportation to bridge sites:

For Howe truss bridges		1.63 cents.
" combination "		2.00 "
" iron and steel "		2.38 "
Castings, finished and delivered.	Howe truss bridges	3.38 "
	Combination "	3.88 "
	Iron and steel "	4.38 "

Timber.

The prices for timber are, for pine, those of the Yellow Pine Lumber Association, with $12 per M. added for freight; and for oak, $18 per M. delivered.

EXAMPLE OF STRAIN CALCULATIONS.

The nearest equivalent uniformly distributed rolling load will be substituted for the wheel loads in the calculation. The results will be nearly the same, and are more readily obtained. The method can be applied to any of the figures given on Plate XXX by aid of Tables 1 and 2.

EXAMPLE, PLATE XXX, CLASS A, FIG. 4.

Combination Bridge. 10 panels of 15'.0 each.

Span, 150'.0 c to c pins. Dead load, 1 220 lbs. per lin. ft. bridge.
Depth, 28'.0 " chords.
Width, 16'.0 " trusses. Rolling load { 2 74-ton cons'n engines, followed by 2 000 lbs. per lin. ft.=2 750 lbs. per lin. ft. bridge.

Truss members.	W = dead load per panel per truss	= 610 × 15 = 9 150 lbs.	Table 1.
	W' = rolling " " " " "	= 1 375 × 15 = 20 625 "	l=150 ft.
	W'' = maximum panel load " "	= 1 930 × 15 = 28 950 "	l= 30 "
	(W + W') horizontal = (9 150 + 20 625) × 15 ÷ 28	= 15 590 "	
Top laterals.	W = wind against one panel of bridge (uniform)	= 72 × 30 = 2 160 lbs.	
Bot. laterals.	W = " " " " " floor (")	= 75 × 15 = 1 125 "	
	W' = " " " " " train (rolling)	= 300 × 15 = 4 500 "	

+ or −	Member.	Dead Load.			Rolling Load.				Max. Vert. Comp.	Ratio.	Max. Strains.	Allowed Strains per □″.	Area Req'd.	Section Used.	Actual Area.
		Panel Load.	Mult. Tab. 2.	Result.	Panel Load	Mult. Tab. 2.	Result.	Imp. %							
	Web.														
+	aB	9 150	4.5	41 170	20 625	4.5	92 810	.....	133 980	1.134	151 930	490	310	2 pcs. 10 × 15	300
−	Be	"	2.25	20 590	"	2.35	48 470	.....	69 060	1.893	130 730	10 000	13.07	2 bars 5 × 1 $\frac{5}{8}$	13.12
	{				{	0.25	5 160	50	0	.....	0				
+	{ eF	"	1.25	11 440	"	1.50	30 940	.....	42 380	1.134	48 060	5 120	9.38	2-10″ Chs	10.5
−	Bc	"	0.75	6 860	28 9.0	0.75	21 710	20	32 910	1.139	37 480	10 000	3.75	2 bars 1 $\frac{3}{8}$″ □	3.78
−	Bd = Fd	"	0.50	4 570	"	0.50	14 470	20	21 930	1.48	32 460	"	3.25	2 " 1 $\frac{1}{16}$″ "	3.44
−	Fc	"	0.25	2 290	"	0.25	7 240	20	10 980	1.918	21 060	"	2.11	2 " 1 $\frac{1}{16}$″ "	2.26
−	Bb = Ff	"	0.50	4 570	"	1.00	28 950	25	.....	.....	40 760	"	4.08	1 bar 2″ □	4.00
+	Cc = Dd, &c.	"	0.50	4 570	.....	.....	00	.....	.....	.....	4 570	183	25	2 pcs. 4 × 8	64
	Chords.														
−	ac = ce	15 950	4.5	.....	.....	.....	.....	.....	.....	.....	71 770	10 000	7.18	2 bars 4 $\frac{1}{2}$″ × 13″	7.31
−	ee	"	12.5	.....	.....	.....	.....	.....	.....	.....	199 370	"	19.94	4 " 4 $\frac{1}{2}$″ × 1 $\frac{1}{2}$″	20.25
+	BDF	"	13.0	.....	.....	.....	.....	.....	.....	.....	207 350	790	262	3 pcs. 6 $\frac{1}{2}$″ × 13 $\frac{1}{2}$″	269
	Portal Bracing—Depth, 8′.5 on aB														
	Tie B′	2 160	4.5	.....	.....	.....	.....	.....	9 700	1.87	18 140	15 000	1.21	1 rod 1 $\frac{1}{4}$″ dia	1.23
	" B	"	4.5	.....	.....	.....	.....	.....	9 700	1.37	13 290	"	0.89	2 rods 1″ "	1.56
	Strut B′B	"	4.5	.....	.....	.....	.....	.....	9 700	4.24	41 130	1 130	37	1 pce. 7″ × 7″	49
	Bottom Lateral Rods.														
	ab	1 125	4.5	5 060	4 500	4.5	20 250	.....	25 310	1.26	31 900	15 000	2.13	1 rod 1 $\frac{5}{8}$″ dia	2.07
	bc	"	3.5	3 990	"	3.6	16 200	.....	20 140	"	25 400	"	1.69	1 " 1 $\frac{1}{2}$″ "	1.77
	cd	"	2.5	2 810	"	2.8	12 600	.....	15 410	"	19 400	"	1.29	1 " 1 $\frac{1}{4}$″ "	1 23
	de	"	1.5	1 690	"	2.1	9 450	.....	11 140	"	14 000	"	.93	1 " 1 $\frac{1}{8}$″ "	.99
	ef	"	0.5	560	"	1.5	6 750	.....	7 310	"	9 200	"	.61	1 " 1″ "	.78
	Bottom Longitudinal Rods.														
	ab	1 125	3.5	3 940	4 500	3.6	16 200	.....	20 140	.94	19 000	15 000	1.27	1 rod 1 $\frac{1}{4}$″ dia	1.23
	bc	"	4.5	5 060	"	4.5	20 250	.....	25 310	"	23 800	"	1.59	1 " 1 $\frac{7}{16}$″ "	1.62
	cd	"	1.5	1 690	"	2.1	9 450	.....	11 140	"	10 500	"	.70	1 " 1″ "	.78
	de	"	2.5	2,810	"	2.8	12 600	.....	15 410	"	14 500	"	.97	1 " 1 $\frac{1}{8}$″ "	.99
	ef	"	0.0	0	"	1.5	6 750	.....	6750	"	6 350	"	.42	1 " 1″ "	.78
	F. Bm	Hanger		4 570	.....	.....	28 950	50	.....	.....	48 000	10 000	4.8	1 bar 2 $\frac{5}{16}$″ □	4.79

Strains and Sections used in the Comparative Estimates of Quantities and Cost for Combination and Howe Truss Bridges.

Combined.—Class A, Fig. 1.

Span, 75'.0 c to c pins.
Depth, 22'.0 " chords.
Width, 15'.4 " trusses.
Dead load, 1 000 lbs. per lin. ft.
Rolling load, two 80.7-ton consolidation engines.
Floor beams, 2 pcs., 9" × 18".
Track stringers, 2 pcs., 8" × 16"; 10' c to c.
Top bracing rods, 1" dia.
Top lateral braces, 6" × 6".
Portal struts, 6" × 12".
Knee braces, 6" × 9".
Bot. end strut, 6" × 7".
Bot. lat. rods, 1⅛" 1". 1". dia.
" long'l " 1". 1". 1¼" dia.
Cross-ties, 7" × 10"—16" c to c.
Guard rail, 5" × 8".
F. Bm. hanger, 1 pce., 2⅛" □.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □"
aB	+	75 900	2	8" × 13"	208
Bd	—	44 800	2	2⅛" × 1"	5.0
Dd	+	29 200	2	6" × 9"	108
Bc = Dc	—	22 600	2	1⅛" □	2.26
Bb	—	37 500	1	1⅛" □	3.75
Cc	+	3 100	2	4" × 7"	56
ad	—	37 500	2	3" × 1⅛"	4.12
BCD	+	75 000	3	5" × 12"	180

Warren.—Class B, Fig. 1.

Bot. long'l rods, all 1" dia.
F. Bm. hanger, 2 pcs., 1⅛" □.
Balance, same as A—1.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □"
aB	+	75 900	2	8" × 13"	208
Bc	—	50 000	2	3" × 1⅛"	5.63
	—	9 100			
cD	+	26 700	2	7" chan's.	6.6
Bb = Dd	—	37 500	2	1⅜" □	3.78
Cc	+	3 100	2	4" × 7"	56
ac	—	37 500	2	3" × 1⅛"	4.12
cc	—	67 500	2	3" × 1¼"	7.50
BD	+	60 000	3	5" × 12"	180

Pratt.—Class C, Fig. 1.

Bot. long'l rods, omitted.
F. Bm. hanger, 2 pcs., 1⅛" □.
Balance, same as A—1.

Class C, Fig. 1—Continued.

Member.		Strain inc. imp.	Section. Pcs.	Size.	Area □"
aB	+	75 900	2	8" × 13"	208
Bc	—	50 000	2	4" × 1⅛"	5.50
Cd	—	26 700	2	2⅛" × ⅝"	3.12
Dc	—	9 100	1	1" □	1.00
Bb	—	37 500	2	1⅜" □	3.78
Cc	+	25 300	2	6⅛" × 8"	104
Dd	+	7 700	2	5" × 8"	80
ac	—	37 500	2	4" × 1⅛"	5.50
cd	—	60 000	2	4" × ⅞"	7.00
CD	+	67 500	3	5" × 12"	180

Howe.—Class D, Fig. 1.

Span, 75'.0 c to c, angle block.
Depth, 23'.0 " chords.
Width, 16' 1" " trusses.
Dead load, 1 450 lbs. per lin. ft.
Rolling load, two 80.7-ton consolidation engines.
Floor beams, 7" × 15"=2' c to c.
Track stringers, 2 pcs., 6" × 12".
Top and bot. bracing rods, 1" dia.
" " lat. braces, 6" × 6".
Cross-ties, 6" × 8"=16" c to c.
Guard rail, 5" × 8".

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □"
aB	+	83 300	2	8⅛" × 9"	153
bC	+	58 600	2	8⅛" × 7"	119
cD	+	36 300	2	8⅛" × 6"	102
dD	+	15 700	2	"	102
dC&c	..	0	1	8" × 6"	48
			4	6" × 10"	240
Aa	..	0	2	1" dia.	1.56
Bb	—	75 500	2	2⅛" dia.	8.4
Cc	—	53 100	2	1⅝" dia.	5.9
Dd	—	32 900	2	1⅞" dia.	3.84
dd	—	70 200	1	22" × 14"	308
DD	+	70 200	1	22" × 10"	220

Combined.—Class A, Fig. 2.

Span, 124'.0 c to c, pins.
Depth, 24'.0 " chords.
Width, 15'.5 " trusses.
Dead load, 1 200 lbs per lin. ft.
Rolling load, two 80.7-ton consolidation engines, followed by 2 240 lin. ft.
Floor beams, 2 pcs., 9" × 18".
Track stringers, 3 pcs., 7" × 16"=11' c to c.

Strains and Sections used in the Comparative Estimates of Quantities, &c.—(*Continued*).

Class A, Fig. 2—Continued.

Top bracing rods, 1″ dia.
Top lateral braces, 6″ × 6″.
Portal struts, 6″ × 12″.
Knee braces, 6″ × 9″.
Bot. end strut, 6″ × 7″.
Bot. lat. rods, 1½. 1⅜. 1⅜. 1″. dia.
" long'l rods, 1½″ 1⅜″ 1″ 1⅛″.
Cross-ties, 7″ × 11″—16″ c to c.
Guard rail, 5″ × 8″.
Floor Bm. hanger, 1 pc. 2¼″ □.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □″
aB	+	139 000	2	8″ ⅛ × 15″	255
Be	—	115 000	2	4″ × 1 7/16″	11.5
Ee	+	71 200	2	6½″ × 13″	169
Bc = Ed.	—	38 100	2	1¾″ □	3.78
Bd = Ec.	—	26 400	2	1 3/16″ □	2.82
Bb	—	44 700	1	2⅛″ □	4.52
Cc = Dd.	+	4 600	2	4″ × 7″	56
ac = ce	—	75 000	2	5″ × ¾″	7.5
BE	+	200 000	1	16″ × 15″	240

Combined.—Class A, Fig. 3.

Bot. long'l rods, 1 3/16″. 1 7/16″. 1⅜″. 1″. dia.
Balance, same as A—2.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □″
aB	+	139 000	2	8⅛″ × 15″	255
Bd	—	104 500	2	4″ × 1 5/16″	10.5
	—	5 700			
Ed	+	44 900	2	9″ chs.	8.6
Bc	—	38 100	2	1¾″ □	3.78
Ec	—	26 400	2	1 3/16″ □	2.82
Bb = Ee	—	44 700	1	2⅛″ □	4.52
Cc = Dd	+	4 600	2	4″ × 7″	56
ad	—	75 000	2	5″ × ¾″	7.5
dd	—	172 000	4	5″ × 7/8″	17.5
BE	+	168 000	1	16″ × 14″	224

Warren.—Class B, Fig. 2.

Floor Bm. hangers, 2 pcs., 1⅛″ □.
Balance, same as A—2.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □″
aB	+	139 000	2	8⅛″ × 15″	255
Bc	—	103 000	2	4″ × 1 5/16″	10.5
cD	+	70 000	2	12″ chs.	13.5
De	+	20 000	1	6″ chs.	12.3
	—	40 200	2	6″ × ¾″	
Bb = Dd	—	44 700	2	1⅛″ □	4.5

Class B, Fig. 2—Con'd.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □″
Cc = Ee	+	4 600	2	4″ × 7″	56
ac	—	75 000	2	5″ × ¾″	7.5
ce	—	161 000	4	5″ × 13/16″	16.25
DF	+	172 000	1	16″ × 14″	224

Pratt.—Class C, Fig. 2.

Bot. long'l rods omitted.
Floor-beam hangers, 2 pcs., 1⅛″ □.
Balance same as A—2.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □″
aB	+	139 000	2	8⅛″ × 15″	255
Bc	—	103 000	2	5″ × 1 5/16″	10.62
Cd	—	70 000	2	4″ × 7/8″	7.0
De	—	40 200	2	3½″ × ¾″	4.37
Ed	—	20 000	2	1″ □	2.00
Dc	..	0	1	1″ "	1.0
Bb	—	44 700	2	1½″ "	4.5
Cc	+	63 500	2	7/8″ × 12″	168
Dd	+	38 400	2	6½″ × 10″	130
Ee	+	17 100	2	6″ × 8″	96
ac	—	75 000	2	5″ × 1⅛″	9.37
cd	—	129 000	2	5″ × 1 5/16″	13.12
dc	—	161 000	4	5″ × 1 3/16″	16.25
DE	+	172 000	1	16″ × 14″	224

Howe.—Class D, Fig. 2.

Span, 124′.0 c to c angle block.
Depth, 23′.5 " chords.
Width, 16′.5 " trusses.
Dead load, 1 750 lbs. per lin. ft.
Rolling load, { two 80.7-ton cons'n engs., followed by 2 240 lin. ft.
Floor beams, 7″ × 15″—2′, c to c.
Track str's, 2 pcs., 6″ × 12″.
Top bracing rods, 1″ dia.
Bot. " " 1½″, 1⅜″, 1 3/16″, 1⅛″, 1″.
Top lateral braces, 6″ × 6″.
Bot. " " 6″ × 7″.
Cross ties, 6″ × 8″—16″ c to c.
Guard rail, 5″ × 8″.

Member.		Strain, inc. imp.	Section. Pcs.	Size.	Area □″
aB	+	151 500	2	11″ × 11″	242
bC	+	125 700	2	11″ × 10″	220
cD	+	101 500	2	11″ × 9″	198
dE	+	77 500	2	11″ × 8″	176
eF	+	55 100	2	11″ × 7″	154

Strains and Sections used in the Comparative Estimates of Quantities, &c.—(*Continued*).

Class D, Fig. 2—Con'd.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
fG	+ 34 400	2	11″ × 6″	132
gF	+ 20 800	1	8″ × 7″	56
fE	+ 1 400	1	8″ × 6″	48
eD, &c.	0	1	8″ × 6″	48
Aa	0	4	7″ × 11″	308
		3	1″ dia.	2.34
Bb	— 136 200	3	$2\frac{7}{16}$″ dia.	14.0
Cc	— 112 700	3	$2\frac{7}{16}$″ "	11.28
Dd	— 90 100	3	$1\frac{13}{16}$″ "	8.85
Ee	— 68 300	3	$1\frac{11}{16}$″ "	6.72
Ff	— 47 700	3	$1\frac{7}{16}$″ "	4.86
Gg	— 33 100	3	$1\frac{3}{16}$″ "	3.33
fg	— 199 800	1	26″ × 15″	390
FG	+ 194 200	1	26″ × 11″	286

Combined.—Class A, Fig. 4.

Span, 147′ .4 c to c pins.
Depth, 28′ .0 " chords.
Width, 16′ .0 " trusses.
Dead load, 1 100 lbs. per lin. ft.
Rolling " 2 500 " " " "
Panel, 64-ton 10-wh. engine.
Floor-beams, 2 pcs., 9″ × 18″.
Track str's, 2 pcs., 9″ × 16′—10′ c to c.
Top bracing rods, 1″ dia.
Top lateral braces, 7″ × 7″.
Portal rods, $1\frac{1}{4}$″ dia.
Portal braces, 7″ × 7″.
Bot. end strut, 6″ × 7″.
Bot. lat. rods, $1\frac{5}{8}$″, $1\frac{7}{16}$″, $1\frac{1}{4}$″, $1\frac{1}{16}$″, 1″ dia.
" long'l " $1\frac{1}{4}$″, $1\frac{1}{16}$″, 1″, $1\frac{1}{16}$″, 1″ "
Cross ties, 7″ × 10″ — 16″ c to c.
Guard rail, 5″ × 8″.
Floor-beam hangers, 1 pce. 2″ □.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
aB	+ 134 900	2	9″ × 15″	285
Be	— 114 900	2	$4\frac{1}{2}$″ × $1\frac{1}{2}$″	11.25
	— 00	2	6″ × 13″	156
eF	+ 42 600	2	9″ chs.	9.0
Bc	— 30 100	2	$1\frac{1}{2}$″ □	3.12
Bd = Fd	— 25 900	2	1″ "	2.54
Fc	— 16 700	2	1″ "	2.00
Bb = Ff	— 32 500	1	$1\frac{13}{16}$″ "	3.29
Cc, &c.	+ 4 100	2	4″ × 8″	64
ac = ce	— 62 800	2	4″ × $1\frac{13}{16}$″	6.5
ee	174 500	4	4″ × $1\frac{3}{16}$″	17.5
BF	+ 181 500	3	6″ × $13\frac{1}{2}$″	243

Combined.—Class A, Fig. 5. Data same as A—4.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
aB	+ 134 900	2	$9\frac{1}{8}$″ × 15″	285
Be	— 109 400	2	5″ × $1\frac{3}{32}$″	10.94
	— 3 840			
eF	+ 40 500	2	9″ chs.	8.6
Bc	— 40 000	2	$1\frac{7}{16}$″ □	4.14
Bd = Fd	— 25 900	2	1″ "	2.54
Bb = Ff	— 32 500	1	$1\frac{13}{16}$″ "	3.29
Cc, &c.	+ 4 100	2	4″ × 8″	64
ac	— 62 800	2	4″ × $\frac{7}{16}$″	6.5
ce	— 76 800	2	4″ × 1″	8.0
ee	174 500	4	4″ × $1\frac{3}{16}$″	17.5
BF	+ 174 500	3	6″ × $13\frac{1}{2}$″	243

Warren.—Class B, Fig. 3.

Depth, 25′ .0 c to c chords.
Portal rods, $1\frac{5}{16}$″ dia.
Floor-beam hangers, 2 pcs., $1\frac{7}{16}$″ □.
Balance same as A—4.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
aB	+ 138 600	2	9″ × 15″	270
Bc	— 109 900	4	4″ × $1\frac{1}{16}$″	11.0
	or	2	8″ × 15″	255
cD	+ 83 400	2	$12\frac{1}{2}$″ chs.	13.8
	+ 1 900	1	6″ ch.	
De	— 59 000	2	5″ × $1\frac{11}{16}$″	9.43
	— 22 000			
eF	+ 36 800	2	9″ chs.	8.6
Bb, &c.	— 32 500	2	$1\frac{13}{16}$″ □	3.44
Cc, &c.	+ 4 100	2	4″ × 8″	64
ac	— 70 400	2	4″ × 7″	7.0
ce	— 164 300	4	4″ × $1\frac{3}{32}$″	16.5
ee	195 600	4	4″ × $1\frac{3}{32}$″	19 5
DF	+ 187 800	3	6″ × 14″	252

Pratt.—Class C, Fig. 3.

Bot. long'l rods omitted.
Balance same as B 3.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
aB	+ 138 600	2	9″ × 15″	270
Bc	— 109 900	2	5″ × $1\frac{1}{4}$″	11.25
Cd	— 83 400	2	4″ × $1\frac{5}{16}$″	8.5
De	— 59 000	2	3″ × 1″	6.0
Ef	— 36 800	2	3″ × $1\frac{1}{16}$″	4.12
Fe	— 22 000	1	$1\frac{1}{4}$″ □	2.25
Ed	— 1 900	1	$\frac{7}{8}$″ "	0.77

Strains and Sections used in the Comparative Estimates of Quantities, &c.—(*Continued*).

Class C, Fig. 3—Con'd.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □″
Dc	..	0	1	$\frac{7}{8}$″ □	0.77
Bb	—	32 500	2	$1\frac{5}{16}$″ “	3.44
Cc	+	76 000	2	7″ × 13″	182
Dd	+	55 000	2	6″ × 12″	144
Ee	+	35 800	2	6″ × 10″	120
Ff	+	18 400	2	6″ × 8″	96
ac	—	70 400	2	5″ × $\frac{7}{8}$″	8.75
cd	—	125 200	2	5″ × $1\frac{1}{4}$″	12.5
de	—	164 300	4	5″ × $1\frac{1}{16}$″	16.25
ef	—	187 800	4	5″ × $1\frac{7}{16}$″	18.75
EF	+	195 600	3	6″ × 14″	252

Howe.—Class D, Fig. 3.

Span, 147′.4 c to c angle block.
Depth, 23′.5 “ chords.
Width, 16′.7 “ trusses.
Dead load, 1 800 lbs. per lin. ft.
Rolling “ 2 500 “ “
Panel “ 64-ton 10-wh. eng.
Floor beams, 7″ × 15″ — 2′ c to c.
Track stringers, 2 pcs. 6″ × 12″
Top bracing rods, 1″ dia.
Bot. “ $1\frac{1}{8}$″, $1\frac{7}{16}$″, $1\frac{6}{8}$″, $1\frac{5}{16}$″, $1\frac{1}{16}$″, 1″
Top and bot. lat. braces, 7″ × 7″
Cross ties, 6″ × 8″ — 16″ c to c.
Guard rail, 5″ × 8″

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □″
aB	+	161 200	2	12″ × 11″	264
bC	+	137 300	2	12″ × 10″	240
cD	+	114 800	2	12″ × 9″	216
dE	+	92 900	2	12″ × 8″	192
eF	+	72 200	2	12″ × 8″	192
fG	+	52 600	2	1 ″ × 6″	144
gH	+	34 000	2	12″ × 6″	144
hG	+	21 800	1	8″ × 7″	56
gF	+	3 700	1	8″ × 6″	48
fE, &c	..	0	1	8″ × 6″	48
Aa	..	0	4	7″ × 12″	336
			3	1″ dia.	2.34
Bb	—	143 900	3	$2\frac{1}{4}$″ dia.	14.73
Cc	—	122 100	3	$2\frac{3}{16}$″ dia.	12.6
Dd	—	101 500	3	$2\frac{1}{16}$″ “	10.65
Ee	—	81 600	3	$1\frac{7}{8}$″ “	8.28
Ff	—	62 700	3	$1\frac{5}{8}$″ “	6.21
Gg	—	44 800	3	$1\frac{3}{8}$″ “	4.44
Hh	—	32 600	3	$1\frac{1}{16}$″ “	3.33
gh	—	248 400	1	28″ × 15″	420
GH	+	243 400	1	28″ × 12″	336

Combined.—Class A, Fig. 6.

Span, 168′.2 c to c pins.
Depth, 28′.0 “ chords.
Width, 16′.0 “ trusses.
Dead load, 1 310 lbs. per lin. ft.
Rolling “ 2 500 “ “
Panel “ 66-ton 10-wh. engine.
Floor beams, 2 pcs. 9″ × 18″
Track str., 2 pcs. 9″ × 16″ — 10′ c to c.
Top bracing rods, 1″ dia.
Top lateral braces, 7″ × 7″
Portal rods, $1\frac{1}{4}$″ dia.
Portal braces, 7″ × 7″
Bot. end strut, 6″ × 7″
Bot. lat. rods, $1\frac{1}{16}$″ to 1″ dia.
“ long'l rods, $1\frac{7}{16}$″ to 1″
Cross ties, 7″ × 10″ — 16″ c to c.
Guard rail, 5″ × 8″
Floor bm. hangers, 1 pce. $2\frac{1}{2}$″ sqr.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □″
aB	+	164 000	2	$11\frac{1}{2}$″ × 15″	345
Bc	—	135 500	4	4″ × $3\frac{1}{2}$″	13.5
cD	+	109 000	2	$10\frac{1}{2}$″ × 15″	310
Dg	—	112 000	2	$4\frac{1}{2}$″ × $1\frac{1}{4}$″	11.25
De = Fe	—	23 300	2	$1\frac{1}{4}$″ □	2.54
Fg	+	60 500	2	9″ × 12″	216
Bb, &c	—	38 400	1	2″ □	4.00
Cc, &c	+	4 600	2	4″ × 8″	64
ac	—	73 400	2	5″ × $\frac{7}{8}$″	7.5
ce = eg	—	180 500	4	5″ × $3\frac{1}{2}$″	18.12
DF	+	267 000	1	23″ × 14″	322
FF	+	240 000	1	22″ × 14″	308

Combined.—Class A, Fig. 7.

Bot. long'l rods, $1\frac{7}{16}$″ to 1″ dia.
Balance same as A–6.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □″
aB	+	164 000	2	$11\frac{1}{2}$″ × 15″	345
Bc	—	135 500	4	4″ × $3\frac{1}{2}$″	13.5
Cc	+	102 300	2	10″ × 14″	280
Cf	—	116 000	2	$4\frac{1}{2}$″ × $1\frac{5}{16}$″	11.81
fG	+	47 100	2	10″ ch's	10.5
Cd	—	34 800	2	$1\frac{5}{16}$″ □	3.44
Ce = Ge	—	29 700	2	$1\frac{1}{4}$″ “	3.12
Gd	—	19 000	2	1″ “	2.00
Bb = Gg	—	38 400	1	2″ “	4.00
Dd, &c	+	4 600	2	4″ × 8″	64
ac	—	73 400	2	5″ × $\frac{7}{8}$″	7.5
cf	—	133 500	2	5″ × $1\frac{5}{16}$″	13.12
ff	—	240 300	4	5″ × $1\frac{7}{16}$″	24.37
CG	+	247 000	1	22″ × 14″	308

Strains and Sections used in the Comparative Estimates of Quantities, &c.—(*Continued*).

Warren.—Class B, Fig. 4.

Depth, 26'.0 c to chords.
Portal rods, $1\frac{3}{16}$'' dia.
Floor bm. hangers, 2 pcs. $1\frac{1}{2}$'' □
Balance same as A-6.

Member.	Strain, inc.	imp.	Pcs.	Section. Size.	Area. □''
aB	+	166 900	2	11'' × 15''	330
Bc	−	138 000	4	4'' × $\frac{7}{8}$''	14.0
cD	....	or	2	9'' × 15''	270
	+	111 000	2	12'' ch's.	18.9
De	−	85 900	4	4'' × $1\frac{1}{16}$''	8.5
eF	−	5 100	1	8'' ch's.	.....
	+	62 000	2	9'' ''	14.55
Fg	+	25 800	1	6'' ''	.....
	−	40 100	2	8'' × $\frac{5}{8}$''	13.3
Bb, &c.	−	38 400	2	$1\frac{7}{16}$'' □	4.14
Cc, &c.	+	4 600	2	4'' × 8''	64
ac	−	79 200	2	5'' × $\frac{13}{16}$''	8.12
ce	−	194 400	4	5'' × $\frac{11}{16}$''	19.37
eg	−	252 000	4	5'' × $1\frac{1}{16}$''	25.0
FF	+	259 200	1	21'' × 15''	315

Pratt.—Class C, Fig. 4.

Bot. long'l rods omitted.
Balance same as B-4.

Member.	Strain, inc.	imp.	Pcs.	Section. Size.	Area. □''
aB	+	166 900	2	11'' × 15''	330
Bc	−	138 000	2	5'' × $1\frac{3}{8}$''	13.75
Cd	−	111 000	2	5'' × $1\frac{1}{8}$''	11.25
De	−	85 900	2	5'' × $\frac{7}{8}$''	8.75
Ef	−	62 000	2	4'' × $\frac{13}{16}$''	6.5
Fg	−	40 100	2	3'' × $\frac{13}{16}$''	4.12
Gf	−	25 800	2	$1\frac{3}{16}$'' □	2.82
Fe	−	5 100	1	1'' ''	1.00
Ed	....	0	1	1'' ''	1.00
Bb	−	38 400	2	$1\frac{7}{16}$'' ''	4.14
Cc	+	102 400	2	9'' × 14''	252
Dd	+	80 200	2	9'' × 12''	216
Ee	+	59 300	2	7'' × 12''	168
Ff	+	39 900	2	7'' × 10''	140
Gg	+	21 900	2	6'' × 9''	108
ac	−	79 200	2	5'' × 1''	10.0
cd	−	144 000	2	5'' × $1\frac{7}{16}$''	14.37
de	−	194 400	4	5'' × 1''	20.0
ef	−	230 400	4	5'' × $1\frac{3}{16}$''	23.12
fg	−	252 000	4	5'' × $1\frac{5}{16}$''	25.62
FG	+	259 200	1	21'' × 15''	315

Howe.—Class D, Fig. 4.

Span, 168'.2, c to c, angle block.
Depth, 23'.5, '' chords.
Width, 17.1 '' trusses.
Dead load, 2 000 lbs. per lin. ft.
Rolling '' 2 500 '' '' ''
Panel '' 66-ton 10-wh. engine.
Floor beams, 7'' × 15''—2', c to c.
Track str's, 2 pcs., 6'' × 12''.
Top bracing rods, 1'' dia.
Bot. '' '' $1\frac{1}{16}$'' to 1'' dia.
Top and bot lateral braces, 7'' × 7''.
Cross ties, 6'' × 8''—16'', c to c.
Guard rail, 5'' × 8''.

Member.	Strain, inc.	imp.	Pcs.	Section. Size.	Area. □''.
aB	+	194 300	2	12'' × 12''	288
bC	+	169 000	2	12'' × 11''	264
cD	+	145 300	2	12'' × 10''	240
dE	+	122 000	2	12'' × 9''	216
eF	+	99 700	2	12'' × 8''	192
fG	+	78 400	2	12'' × 7''	168
gH	+	57 900	2	12'' × 7''	168
hI	+	38 300	2	12'' × 6''	144
iH	+	25 800	1	12'' × 6''	72
hG	+	6 200	1	''	72
gF, &c.	..	0	1	''	72
Aa	..	0	4	7'' × 12''	336
			3	1'' dia.	2.34
Bb	−	173 800	3	$2\frac{1}{2}$'' dia.	17.82
Cc	−	151 000	3	$2\frac{1}{16}$'' ''	14.73
Dd	−	129 100	3	$2\frac{5}{16}$'' ''	13.29
Ee	−	107 800	3	$2\frac{1}{16}$'' ''	10.65
Ff	−	87 500	3	$1\frac{13}{16}$'' ''	8.85
Gg	−	68 000	3	$1\frac{11}{16}$'' ''	6.72
Hh	−	49 300	3	$1\frac{7}{16}$'' ''	4.86
Ii	−	36 600	3	$1\frac{1}{16}$'' ''	3.69
hi	−	338 600	1	33'' × 16''	528
HI	+	333 300	1	33'' × 12''	396

Combined.—Class A, Fig. 8.

Span, 210'.0, c to c pins.
Depth, 28'.0 '' chords.
Width, 16'.5 '' trusses.
Dead load, 1 650 lbs. per lin. ft.
Rolling '' 2 500 '' '' ''
Panel '' 66-ton 10-wh. engine.
Floor beams, 2 pcs., 9'' × 18''.
Track str's, 3 pcs., 7'' × $\frac{1}{16}$''. 11'.5, c to c.
Top bracing rods, $1\frac{1}{16}$'' to 1'' dia.
Top lateral braces, 7'' × 7''.

Strains and Sections used in the Comparative Estimates of Quantities, &c.—(*Continued*).

Class A, Fig. 8—Con'd.

Portal rods, $1\frac{1}{2}$" dia.
" braces, 7" × 8".
Bot. end strut, 7" × 7".
Bot. lat. rods, 2" to 1" dia.
" long'l rods, $1\frac{1}{16}$" to 1" dia.
Cross ties, 7" × 11" — 16" c to c.
Guard rail, 5" × 8".
Floor bm. hangers, 1 pce, $2\frac{3}{16}$" □.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □".
aB	÷	229 600	3	10" × 16"	480
Bc	—	195 900	4	5" × 1"	20.0
		or	2	12" × 17"	408
cD	+	163 700	2	15" chs.	26.0
Dg	—	149 200	2	6" × $1\frac{1}{4}$"	15.0
	—	2 400			
gH	+	60 200	2	10" chs.	11.7
De	—	39 300	2	$1\frac{7}{16}$" □	41.4
Df = Hf	—	34 000	2	$1\frac{5}{16}$" "	3.44
He	—	21 800	2	$1\frac{1}{16}$" "	2.26
Bb & c	—	41 200	1	$2\frac{7}{16}$" "	4.25
Cc & c	+	6 200	2	4" × 8"	64
ac	—	108 400	2	5" × $1\frac{1}{4}$"	11.25
Ce = eg	—	275 200	4	5" × $1\frac{3}{4}$"	27.5
eg	—	408 700	6	5" × $1\frac{3}{4}$"	41.25
DH	+	417 000	3	10" × 17"	510

Warren.—Class B, Fig. 5.

Floor bm. hangers, 2 pcs., $1\frac{5}{16}$" □.
Balance, same as A–8.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □.
aB	+	229 600	3	10" × 16"	480
Bc	—	195 900	4	5" × 1"	20.0
		or	2	12" × 17"	408
cD	+	163 700	2	15" chs.	26.0
De	—	132 900	2	5" × $1\frac{3}{4}$"	13.75
eF	+	103 500	2	12" chs.	18.3
	+	7 400	1	7" ch.	
Fg	—	75 800	2	6" × $\frac{3}{4}$"	13.2
	—	32 800			
gH	+	49 500	2	10" Chs.	10.5
Bb, &c	—	41 200	2	$1\frac{7}{16}$" □	4.14
Cc, &c	+	6 200	2	4" × 8"	64
ac	—	108 400	2	5" × $1\frac{1}{4}$"	11.25
ce	—	275 200	4	5" × $1\frac{3}{4}$"	27.5
eg	—	375 100	6	5" × $1\frac{3}{4}$"	37.5
FH	+	400 100	3	10" × 16"	480

Whipple.—Class E, Fig. 2.

Bot. long'l rods omitted.
Floor bm. hangers, pcs., $1\frac{5}{16}$" □.
Balance, same as A—8.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □".
aB	+	229 600	3	10" × 16"	480
Bc	—	106 700	2	5" × $1\frac{3}{4}$"	10.62
Bd	—	115 100	2	5" × $1\frac{15}{16}$"	11.87
Ce	—	96 500	2	5" × 1"	10.0
Df	—	75 400	2	$4\frac{1}{2}$" × $\frac{7}{8}$"	7.87
Eg	—	58 500	2	4" × $\frac{3}{4}$"	6.0
Fh	—	42 400	2	$2\frac{1}{2}$" × $\frac{7}{8}$"	4.37
Gg	—	30 600	2	$1\frac{1}{2}$" □	3.12
Hf	—	11 900	2	$\frac{7}{8}$" "	1.53
Ge	..	0	2	"	1.53
Bb	—	41 200	2	$1\frac{7}{16}$" □	4.14
Cc	+	71 900	1	13" × 15"	195
Dd	+	57 600	1	12" × 14"	168
Ee	+	46 100	1	12" × 12"	144
Ff	+	35 100	1	11" × 11"	121
Gg	+	27 100	1	10" × 10"	100
Hh	+	11 400	1	8" × 10"	80
ac	—	108 400	2	5" × $1\frac{1}{4}$"	13.12
cd	—	158 400	4	5" × $1\frac{13}{16}$"	16.25
de	—	241 700	4	5" × $1\frac{7}{16}$"	24.37
ef	—	308 700	4	5" × $1\frac{9}{16}$"	31.25
fg	—	358 400	6	5" × $1\frac{3}{16}$"	35.63
gh	—	391 700	6	5" × $1\frac{7}{16}$"	39.37
FH	+	408 700	1	30" × 16"	480

Howe.—Class D, Fig. 5.

Span, 210'.0, c to c, angle block.
Depth, 28'.0 " chords.
Width, 18'.1 " trusses.
Dead load, 2 400 lbs. per lin. ft.
Rolling " 2 500" " "
Panel " 66-ton 10-wh. engine.
Floor beams, 7" × 15" — 2', c to c.
Track stringers, 2 pcs., 6" × 12".
Top bracing rods, $1\frac{1}{2}$" to 1" dia.
Bot. " " $1\frac{1}{8}$" to 1" dia.
Top and bot. lat. braces, 7" × 7"
Cross ties. 6" × 8" — 16", c to c.
Guard rail, 5" × 8".

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area □".
aB	+	260 000	2	14" × 13"	364
bC	+	234 000	2	14" × 12"	336
cD	+	210 000	2	14" × 12"	336
dE	+	185 500	2	14" × 11"	308

Strains and Sections used in the Comparative Estimates of Quantities, &c.—(*Continued*).

Howe.—Class D, Fig. 5—Con'd.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
eΓ.....	+ 162 000	2	14″ × 10″	280
fG.....	+ 139 500	2	14″ × 9″	252
gH.....	+ 117 000	2	14″ × 8″	224
hI.....	+ 95 500	2	14″ × 8″	224
iK.....	+ 74 500	2	14″ × 7″	196
kL.....	+ 54 400	2	14″ × 6″	168
lL.....	+ 34 800	2	14 × 6″	168
lK.....	+ 22 800	1	12″ × 6″	72
kI.....	+ 2 900	1	12″ × 6″	72
iH, &c....	 0	1	"	72
Aa.....	 0	6	8″ × 12″	576
		4	1″ dia.	3.12

Howe.—Class D, Fig. 5—Con'd.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area □″
Bb.....	— 241 000	4	$2\frac{3}{4}$″ dia	23.76
Cc.....	— 217 000	4	$2\frac{5}{8}$″ "	21.64
Dd.....	— 194 000	4	$2\frac{5}{8}$″ "	19.64
Ee.....	— 171 000	4	$2\frac{5}{8}$″ "	17.03
⁺f.....	— 149 000	4	$2\frac{3}{16}$″ dia.	15.04
Gg.....	— 127 400	4	$2\frac{1}{2}$″ "	12.56
Hh.....	— 106 500	4	$1\frac{7}{8}$″ "	11.04
Ii.....	— 86 200	4	$1\frac{11}{16}$″ "	8.96
Kk.....	— 66 400	4	$1\frac{7}{8}$″ "	6.48
Ll.....	— 47 300	4	$1\frac{1}{2}$″ "	4.92
ll.....	— 481 300	1	$42\frac{1}{2}$″ × 16″	680
LL.....	+ 481 300	1	$42\frac{1}{2}$″ × 14″	595

Strains and Sections used in the Comparative Estimates of Quantities and Cost for Iron and Steel Bridges.

Combined.—Class A, Fig. 1.

Vertical posts omitted.
Span, 93′ .5 c to c pins.
Depth, 22′ .5 c to c chords.
Width, 15′ .5 " trusses.
Dead load, 1 050 lbs. per lin. ft.
Rolling load, two 82.5-ton cons. engs.
Floor beams, 26″ deep, area = 22.83″ □.
Track strs., 20″ " " 19.45 "
Top struts, 2–5″ chans. = 4.62″ □.
Top lateral rods, 1″ dia.
Portal struts, 2.6″ chans. = 5.22 ″ □.
Bot. end strut, 2.5″ " = 4.62 ″ □.
Bot. lateral rods, $1\frac{1}{2}$″ .$1\frac{1}{4}$″ .1″ dia.
Bot. long'l " $1\frac{1}{8}$″ .$1\frac{1}{16}$″ .$1\frac{1}{16}$″ "
Floor bm. hangers, .1 loop .$1\frac{1}{16}$″ □.
Cross ties, 7″ × 10″—16″ c to c.
Guard rail, 5″ × 8″.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
Iron.				
aB.....	+ 98 000	1	9″ ch.	
		2	12″ chs.	17.85
Bd.....	— 60 000	2	3″ × 1″	6 00
Dd.....	+ 33 200	2	7″ chs.	6.54
Bc = Dc..	— 24 900	2	$1\frac{1}{8}$″ □	2.54
bb.....	— 42 700	2	3″ × $\frac{3}{4}$	4 50
ad.....	— 55 700	2	$3\frac{1}{2}$″ × $\frac{13}{16}$″	5.68
BD	+ 111 500	1	9″ ch.	
		2	12″ chs.	19.17

Warren—Class B, Fig. 1.

Bot. long'l rods, $1\frac{1}{2}$″ .$1\frac{1}{4}$″ .1″ dia.
Balance same as A—1.

Member.	Strain. inc. imp.	Section. Pcs.	Size.	Area □″
Iron.				
aB.....	+ 98,000	1	9″ ch.	
		2	12″ chs.″	17.85
Bc.....	— 63 400	2	$3\frac{1}{2}$″ × $\frac{15}{16}$″	6.56
cD.....	— 12 600	1	6″ ch.	
	+ 34 400	2	$7\frac{1}{2}$″ × $\frac{5}{8}$″	11.98
Bb = Dd..	— 42 700	2	3″ × $\frac{3}{4}$″	4.5
ac.....	— 55 700	2	$3\frac{1}{2}$″ × $\frac{13}{16}$″	5.68
cc.....	— 100 400	2	$3\frac{1}{2}$″ × $1\frac{7}{16}$″	10.06
BD.....	+ 89 200	1	9″ ch.	
		2	12″ chs.	17.85

Pratt —Class C, Fig. 1.

Bot. long'l rods omitted.
Balance same as A—1.

STRAINS AND SECTIONS USED IN THE COMPARATIVE ESTIMATES OF QUANTITIES, &c.—(*Continued*).

Pratt.—Class C, Fig 1—Con'd.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □″
Iron.					
aB	+	98 000	1	9″ ch.	
			2	10″ chs.	18.93
Bc	−	63 400	2	$3\frac{1}{2}$″ × $\frac{13}{16}$″	6.56
Cd	−	34 400	2	$2\frac{1}{2}$″ × $\frac{7}{8}$″	3.75
Dc	−	12 600	1	$1\frac{1}{4}$″ □	1.27
Bb	−	42 700	2	3″ × $\frac{7}{8}$″	4.5
Cc	+	28 400	2	7″ chs.	6.3
Dd	+	7 500	2	7″ “	6.3
ac	−	55 700	2	4″ × $\frac{7}{8}$″	7.0
cd	−	89 200	2	4″ × $1\frac{1}{8}$″	9.0
BC	+	89 200	1	9″ ch.	
			2	10″ chs.	14.85
CD	+	100 400	....	do.	14.85

Combined.—Class A, Fig. 5.

Span, 150′ .0 c to c pins.
Depth, 26′ .0 “ chords.
Width, 16′ .0 “ trusses.
Dead load, 1 220 lbs. per lin. ft.
Rolling “ two 74-ton cons. engs., followed by 2 000 lbs. per ft.
Floor bms. 24″ deep. Area = 22 2□″
Track strs. 18″ “ “ 10.43□″
Top struts, 4 angles, 2″ × $2\frac{1}{2}$″ = 4.2□″
Top lat. rods 1″ □.
Portals latticed
Bot. lat. rods, $1\frac{1}{2}$″ to 1″ □.
Bot. long'l “ omitted.
Floor bm. hangers, 8″ × $\frac{13}{16}$″.
Cross ties, 7″ × 10″—16′ c to c.
Guard rail, 5″ × 8″.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area □″
Iron.					
aB	+	154 600	2	12″ ch's.	26.82
Bc	−	131 500	2	5″ × $1\frac{5}{16}$″	13.12
Fe	−	4 300			
	+	46 300	2	10″ chs.	9.6
Bc	−	50 700	2	$2\frac{1}{2}$″ × 1″	5.0
Bd = Fd	−	34 000	2	$2\frac{1}{2}$″ × $1\frac{1}{16}$″	3.43
Bb = Ff	−	40 800	2	$2\frac{1}{2}$″ × $1\frac{1}{16}$″	4.06
Cc, &c.	+	4 600	2	4″ chs.	3.6
ac	−	77 300	2	5″ × $\frac{13}{16}$″	8.12
ce	−	94 500	2	5″ × $\frac{15}{16}$″	9.37
ee	−	214 700	4	5″ × $1\frac{3}{32}$″	21.87
BF	+	214 700	2	12″ chs.	26.82

Warren.—Class B, Fig. 3.

Data same as A-5.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □″
Iron.					
aB	+	154 600	2	12″ chs.	26.82
Bc	−	122 600	2	5″ × $1\frac{1}{4}$″	12.5
cD	+	93 000	2	12″ chs.	16.02
	+	2 100	1	6″ chs.	
De	−	65 600	2	5″ × $\frac{13}{16}$″	10.67
	−	23 200			
eF	+	41 000	2	10″ chs.	9.6
Bb & c	−	40 800	2	$2\frac{1}{2}$″ × $\frac{13}{16}$″	4.06
Cc & c	+	4 600	2	4″ chs.	3.6
ac	−	77 300	2	5″ × $\frac{13}{16}$″	8.12
ce	−	180 400	4	5″ × $\frac{15}{16}$″	18.12
ee	−	214 700	4	5″ × $1\frac{3}{32}$″	21.87
BD	+	137 400	2	12″ chs.	18.00
DF	+	206 200	2	12″ “	25.80

Pratt.—Class C, Fig. 3.

Bot. lat. rods, $1\frac{1}{4}$″ to 1″ □
Floor beams riveted to posts.
Balance same as A-5.

Member.	Strain, inc. imp.		Section. Pcs.	Size.	Area. □″
Iron.					
aB	+	154 600	2	12″ chs.	26.81
Bc	−	122 600	2	5″ × $1\frac{1}{4}$″	12.5
Cd	−	93 000	2	5″ × $\frac{15}{16}$″	9.37
De	−	65 600	2	3″ × $1\frac{1}{16}$″	6.75
Ef	−	41 000	2	3″ × $\frac{13}{16}$	4.12
Fe	−	23 200	2	$1\frac{1}{2}$″ □	2.54
Ed	−	2 100	2	$\frac{7}{8}$″ □	1.54
Bb	−	40 800	2	$2\frac{1}{2}$″ × $\frac{13}{16}$″	4·06
Cc	+	96 000	2	10″ chs.	14.4
Dd	+	68 600	2	10″ “	10.5
Ee	+	44 000	2	8″ “	7.5
Ff	+	21 100	2	6″ “	5.1
ac	−	77 300	2	5″ × $\frac{13}{16}$″	9.37
cd	−	137 400	4	5″ × $\frac{13}{16}$″	13.75
de	−	180 400	4	5″ × $\frac{15}{16}$″	18.12
ef	−	206 200	4	5″ × $1\frac{3}{32}$″	20.62
BC	+	137 400	2	12″ chs.	18.00
CD	+	180 400	2	12″ “	22.5
DE	+	206 200	2	12″ “	25.8
EF	+	214 800	2	12″ “	27.0

Strains and Sections used in the Comparative Estimates of Quantities, &c.—(*Continued*).

Whipple.—Class E, Fig. 1.

Data same as C-3.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
Iron.				
aB	+ 154 600	2	12″ chs.	26.82
Bc	— 71 000	2	3½″ × 1″	7.00
Bd	— 70 000	2	3½″ × 1″	7.00
Ce	— 53 500	2	3½″ × 1 1/16″	5.68
Df	— 33 800	2	2½″ × 1 1/16″	3.43
Ee	— 25 600	2	1½″ □	2.54
Fd	— 6 700	2	⅞″ □	1.54
Ec	.. 0	2	⅞″ □	1.54
Bb	— 40 800	2	2½″ × 1 1/16″	4.06
Cc	+ 38 000	2	7″ chs.	6.6
Dd	+ 25 100	2	6″ "	5.1
Ee	+ 16 400	2	6″ "	5.1
Ff	+ 6 300	2	6″ "	5.1
ac	— 77 300	2	5″ × 1 1/16″	9.37
ce	— 111 500	2	5″ × 1½″	11.25
de	— 163 000	4	5″ × 1 1/16″	16.25
ef	— 197 500	4	5″ × 1″	20.0
BC	+ 163 000	2	12″ chs.	20.4
CD	+ 197 500	2	12″ "	24.6
DF	+ 214 800	2	12″ "	27.0

Combined.—Class A, Fig. 13.

Span, 520′.0 c to c pins.
Depth, 65′.0 " chords.
Width, 22′.0 " trusses.
Dead load, 3 700 lbs. per lin. ft.
Rolling load, two 80.7-ton consolidation engines, followed by 2 240 lin. ft.
Floor bms. 36″ deep. Area=45.21 □″.
Track stringers, 30″ deep. Area=32.07 □″.
Top struts, 4 angles, 3″ × 4″=9 96 □″.
Top lat. rods, 5.7″ □ to 2.0 □″.
Sub struts, 4 angles, 3″ × 3″ = 7.08 □″.
Diagonal rods, 1⅛″ dia.
Portals, latticed.
Bot. end strut, two 10″ chs = 13.8 □″.
Bot. lat. rods, 11.25 □″ to 3.12 □″.
Center long'l strut, two 6″ chs = 5.1 □″.
Floor beam hanger, 8″ × 1 1/16″.
Bot. long'l rods, omitted.
Cross-ties, 8″ × 10″ = 16″ c to c.
Guard rails, 6″ × 8″ and 8″ × 12″.
Bottom laterals, } Steel.
Floor bm. hangers, } Steel.
Balance, Iron.

Class A, Fig. 13—Continued.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
Steel.				
aB	+ 986 200	1	42″ × 22″	107.0
BC	+ 934 300	1	"	101.2
Cf	— 789 500	4	7″ × 2″	56.0
Gf	+ 483 300	1	30″ × 16″	32.4
Gk	— 334 700	4	6″ × 1″	24.0
Lk	+ 182 000	1	18″ × 15	21.0
Cd′=Gh′	— 157 700	2	5″ × 1 5/16″	11.25
Gf′=Lk′	— 113 000	2	4″ × 1″	8.0
h′i=k′i, d′e=f′e,	— 97 700	2	4″ × ⅞″	7.0
Cc	— 122 700	2	4″ × 1 5/16″	9.0
Ed′ = Ih	— 67 500	2	3″ × 1 1/16″	4.87
Bb, &c	— 81 500	2	3″ × 1″	6.0
Gg = Ll	— 81 500	2	"	6.0
abc	— 616 400	4	7″ bars.	44.72
cdef	— 583 900	4	7″ "	42.00
fghik	— 1 362 500	8	7″ "	97.12
kk	— 1 622 000	8	7″ "	116.4
CDE	+ 1 270 900	1	42″ × 22″	90.8
EFG	+ 1 248 000	1	"	89.4
GHI	+ 1 660 100	1	"	118.6
IKL	+ 1 637 300	1	"	117.0
Iron.				
Bc	+ 66 000	2	10″ chs.	12.3
Ee = Ii	+ 85 400	2	"	15.4
Hh′=Kk′, Dd′=Ff′,	+ 24 000	2	7″ chs.	6.6
Ef = Ik′	— 15 400	2	⅞″ □	1.54

Whipple.—Class E, Fig. 3.

Data same as A-13.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
Steel.				
aB	+ 986 200	1	42″ × 22″	107.0
BC	+ 934 300	1	"	101.2
Cd	— 350 200	4	5″ × 1 5/16″	25.0
Ce	— 368 600	4	5″ × 1 13/16″	26.25
Df	— 318 800	4	5″ × 1 1/16″	22.5
Eg	— 273 200	4	5″ × 1″	20.0
Fh	— 225 500	4	3½″ × 1 13/16″	16.62
Gi	— 182 100	4	3½″ × 1 11/16″	13.12
Hk	— 136 600	4	3½″ × 1 1/16″	9.62
Il	— 101 500	2	3½″ × 1 1/16″	7.42
Bb	— 81 500	2	3″ × 1″	6.00
Cc	— 122 700	2	4″ × 1 5/16″	9.00
abc	— 616 400	4	7″ bars.	44.72

STRAINS AND SECTIONS USED IN THE COMPARATIVE ESTIMATES OF QUANTITIES, &c.—(*Continued*).

Whipple.—Class E, Fig. 3—Con'd.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
Steel.				
cd.....	— 583 900	4	7″ bars.	42.00
de.....	— 713 700	6	7″ "	51.63
ef.....	— 940 800	6	7″ "	67.38
fg.....	— 1 135 400	6	7″ "	81.37
gh.....	— 1 297 600	8	7″ "	92.75
hi.....	— 1 427 400	8	7″ "	102.4
ik.....	— 1 524 700	8	7″ "	108.5
kl.....	— 1 589 600	8	7″ "	113.8
CD.....	+ 940 800	1	42″ × 22″	81.1
DE.....	+ 1 135 400	1	"	81.1
EF.....	+ 1 297 600	1	42″ × 22″	92.7
FG.....	+ 1 427 400	1	"	102.1
GH.....	+ 1 524 700	1	"	109.0
HI.....	+ 1 589 600	1	"	113.5
IK=KL...	+ 1 622 000	1	"	115.9

Whipple.—Class E, Fig. 3—Con'd.

Member.	Strain, inc. imp.	Section. Pcs.	Size.	Area. □″
Iron.				
Bc.....	+ 66 000	2	10″ chs.	12.3
Dd.....	+ 273 100	6	10″ "	60.3
Ee.....	+ 237 400	6	10″ "	52.2
Ff.....	+ 200 200	6	10″ "	40.8
Gg.....	+ 166 300	4	10″ chs.	30.0
Hh.....	+ 130 700	4	10″ "	24.0
Ii.....	+ 98 300	2	10″ "	17.7
Kk.....	+ 64 400	2	10″ "	11.4
Ll.....	+ 37 200	2	8″ "	7.8
Kk.....	— 64 600	2	1⅝″ □	6.58
Li.....	— 23 400	2	1¼″ "	2.54
Kh.....	— 0	2	1″ "	2.00
Ig.....	.. 0	2	1″ "	2.00

SUMMARIES OF QUANTITIES AND COST FOR 520 FT. SPANS.

Combined.—Class A, Fig. 13.

Steel	1 132 977 lbs.	@ 6.38 cents	=	$72 283.93
Wrought-iron	374 403 "	" 5.66 "	=	21 191.21
Cast-iron	5 800 "	" 4.38 "	=	254.04
Timber (W. O.)	49 600 B. M.	" 18.00 "	=	892.80
Total cost				$94 621.98
Cost per lineal foot				$181.96

Whipple.—Class E, Fig. 3.

Steel	1 084 554 lbs.	@ 6.38 cents	=	$69 194.55
Wrought-iron	598 578 "	" 5.91 "	=	35 375.96
Cast-iron	5 800 "	" 4.38 "	=	254.04
Timber (W. O.)	49 600 B. M.	" 18.00 "	=	892.80
Total cost				$105 717.35
Cost per lineal foot				$203.30

NOTE.—The weight of structure assumed in the preceding calculation of strains and sections was found to be somewhat too high. The resulting weights, allowing 3 per cent. waste in manufacture, were in favor of stability to the extent of 84 pounds per lineal foot for the Whipple truss, and 323 pounds per lineal foot for the Combined truss.

A reduction in the weight of the steel members of the Combined truss has been made in the above summary sufficient to reduce this estimate to the same basis as the Whipple, viz., 84 pounds per lineal foot in favor of stability.

COMPARATIVE ECONOMY OF IRON AND COMBINATION BRIDGES.

We will consider a span of 150 feet c to c pins, Class A, Figure 5, proportioned for a rolling load of two 74-ton consolidation engines, followed by 2 000 pounds per lineal foot freight load, equivalent to 2 750 pounds per lineal foot of bridge.

Dead load, 1 220 pounds per lineal foot of bridge.

Depth of truss, 26 feet.

The quantities of material are the results of careful detail estimates. The prices of material are those previously given. The prices for erection and renewals are those of an experienced contractor for such work, who would execute at the prices named without delay to trains, and the estimated average life of timber has been furnished by the Superintendent of Bridges, L. & N. R. R.

Specifications preceding. Interest, 6 per cent.

I.—*Iron Bridge with Cross Iron Floor Beams, and Iron Stringers* 10'.0 *c to c.*

Wrought-iron	918.1 lbs.	@	5.80c.	= $53.25	
Cast-iron	7.3 "	"	4.38c.	= 0.32	
Timber—Y. pine	6'.9 B. M.	"	2.7c.	= 0.19	
" W. oak	59'.7 "	"	1.8c.	= 1.07	
Framing and raising				= 8.50	
Total first cost, per lineal foot				= $63.33	$63.33
Every 2 years, painting				1.00 ÷ .1236 =	8.09
" 7 " timber—material			= $1.26		
" 7 " " labor			= 1.00 =	2.26 ÷ .5033 =	4.50
Total ultimate cost, per lineal foot					= $75.92

II.—*Iron Bridge with Cross Iron Floor Beams and Wooden Stringers.*

Wrought-iron	773.5 lbs.	@	5.87c.	= $45.40	
Cast-iron	10.8 "	"	4.38c.	= 0.47	
Timber—Y. pine	65'.2 B. M.	"	3.28c.	= 2.14	
" W. oak	59'.7 "	"	1.8c.	= 1.07	
Framing and raising				= 8.50	
Total first cost, per lineal foot				= $57.58	$57.58
Every 2 years, painting				0.90 ÷ .1236 =	7.28
" 7 " timber—material			= $3.21		
" 7 " " labor			= 1.25 =	4.46 ÷ .5033 =	8.86
Total ultimate cost, per lineal foot					= $73.72

III.—*Iron Bridge with Longitudinal Iron Floor Beams and Long Cross-Ties.*

Wrought-iron	827.4 lbs.	@	5.84c.	= $48.32	
Cast-iron	13.4 "	"	4.38c.	= 0.59	
Timber—Y. pine	6'.9 B. M.	"	2.7c.	= 0.19	
" W. oak	126'.7 "	"	1.8c.	= 2.28	
Framing and raising				= 8.00	
Total first cost, per lineal foot				= $59.38	$59.38

TABLE No. 3.

Comparative Estimates for Combination Through Bridges. Summaries of Quantities and Cost.

Plate I. Class.	Plate I. Fig.	Span c to c Pins.	No. Panels.	Kind of Truss.	Material. Wro't Iron. Lbs.	Wro't Iron. Rate.	Cast Iron. Lbs.	Cast Iron. Rate.	Y. Pine. Ft. B.M	Y. Pine. Rate.	W. Oak. Ft. B.M.	W. Oak. Rate.	Cost per Lin. Ft. Iron Work.	Cost per Lin. Ft. Entire Work Del'd.
A	1	75' .0	6	Combined	11 725	5.11	6 351	3.88	11 630	2.95	4 550	1.8	$11.26	$16.93
B	1	"	6	Warren	14 161	5.30	6 188	"	11 250	"	4 550	"	13.21	18.72
C	1	"	6	Pratt	12 372	5.13	7 403	"	11 970	"	4 550	"	12.29	18.09
D	1	"	7	Howe (clamps wood)	10 048	4.87	6 681	3.38	25 680	3.0	3 180	"	9.53	20.56
"	1	"	7	" (" iron)	10 780	4.86	7 973	"	25 680	"	2 900	"	10.57	21.54
A	2	124' .0	8	Combined	31 161	5.05	8 917	3.88	21 540	3.5	8 720	1.8	15.48	22.82
A	3	"	8	Combined	33 108	5.19	8 678	"	20 630	"	8 720	"	16.56	23.66
B	2	"	8	Warren	38 329	5.29	9 500	"	20 440	"	8 720	"	19.32	26.36
C	2	"	8	Pratt	31 721	5.09	11 670	"	22 420	"	8 720	"	16.66	24.26
D	2	"	12	Howe (clamps wood)	23 869	4.93	16 615	3.38	46 800	3.3	7 140	"	14.01	27.50
"	"	"	12	" (" iron)	26 554	4.89	20 761	"	46 800	"	6 340	"	16.12	29.50
A	4	147' .4	10	Combined (eF wood)	35 062	5.07	12 600	3.88	25 780	3.12	8 840	1.8	15.37	21.92
A	4	"	10	" (" iron)	39 773	5.17	12 000	"	24 100	"	8 840	"	17.10	23.29
A	5	"	10	" (" iron)	38 909	5.16	12 000	"	24 100	"	8 840	"	16.78	22.97
B	3	"	10	Warren (cD wood)	40 321	5.21	13 000	"	25 940	"	8 840	"	17.67	24.24
B	3	"	10	" (" iron)	47 744	5.33	12 120	"	23 430	"	8 840	"	20.45	26.50
C	3	"	10	Pratt	39 314	5.09	16 350	"	26 360	"	8 840	"	17.88	24.54
D	3	"	14	Howe (clamps wood)	29 249	4.88	19 426	3.38	58 640	3.35	7 900	"	14.13	28.42
D	3	"	14	" (" iron)	33 169	4.85	24 511	"	58 640	"	6 850	"	16.52	30.69
A	6	168' .2	12	Combined	52 545	5.08	15 853	3.88	35 150	3.25	9 980	1.8	19.52	27.38
A	7	"	12	Combined	58 877	5.15	14 743	"	32 420	"	9 980	"	21.42	28.76
B	4	"	12	Warren (cD wood)	61 951	5.22	15 682	"	32 015	"	9 980	"	22.84	30.09
B	4	"	12	" (" iron)	71 648	5.32	14 822	"	29 315	"	9 980	"	26.08	32.81
C	4	"	12	Pratt	61 008	5.09	20 553	"	34 900	"	9 980	"	23.20	31.01
D	4	"	16	Howe (clamps wood)	38 753	4.92	25 821	3.38	74 790	3.5	7 870	"	16.51	32.92
D	4	"	16	" (" iron)	43 726	4.89	31 760	"	74 790	"	6 640	"	19.07	35.35
A	8	210' .0	14	Combined {cD wood / gH iron}	96 362	5.12	21 245	3.88	50 240	3.45	14 090	1.8	27.41	36.93
A	8	"	14	" {cD iron / gH " }	110 386	5.25	19 543	"	45 890	"	14 090	"	31.18	39.93
B	5	"	14	Warren... {cD wood / eF iron}	105 740	5.26	22 222	"	48 770	"	14 090	"	30.59	39.81
B	5	"	14	" {cD iron / eF " }	119 764	5.37	20 520	"	44 420	"	14 090	"	34.41	42.92
E	2	"	14	Whipple	95 792	5.06	26 862	"	49 960	"	14 090	"	28.04	37.45
D	5	"	21	Howe (clamps wood)	78 522	4.87	47 100	3.38	120 690	3.55	10 960	"	25.31	46.66
D	5	"	21	" (" iron)	83 595	4.85	55 255	"	120 690	"	9 200	"	28.19	49.38
				Iron Bridges.										
A	1	93' .5	6	Combined (V. Post omitted)	62 889	5.80	550	4.38	666	3.6	5 612	1.8	39.27	40.60
B	1	"	6	Warren (do)	64 330	5.78	550	"	666	3.6	5 612	"	40.03	41.36
C	1	"	6	Pratt	65 923	5.80	550	"	666	3.6	5 612	"	41.15	42.48
A	5	150' .0	10	Combined	137 721	5.80	1 100	4.38	1 040	2.7	8 950	1.8	53.57	54.83
B	3	"	10	Warren	140 503	5.81	1 100	"	1 040	"	8 950	"	54.74	56.00
C	3	"	10	Pratt	149 169	5.79	1 100	"	1 040	"	8 950	"	57.90	59.16
E	1	"	10	Whipple	141 508	5.77	1 100	"	1 040	"	8 950	"	54.75	56.01

Every 2 years, painting			0.90 ÷ .1236 =	7.28
" 7 " timber—material		= $2.47		
" 7 " " labor		= 1.50 = 3.97 ÷ .5033 =		7.88
Total ultimate cost, per lineal foot				= $74.54

IV.—*Combination Bridge with Cross Wooden Floor Beams, and Wooden Stringers 10′.0 c to c. Intermediate Braces Iron. Not Roofed.*

Wrought-iron	307.7 lbs.	@ 5.19c. =	$15.97	
Cast-iron	80.0 "	" 3.88c. =	3.10	
Timber—Y. pine	177′.3 B. M.	" 3.15c. =	5.58	
" W. oak	59′.7 "	" 1.8c. =	1.07	
Framing and raising		=	8.50	
Total first cost, per lineal foot			$34.22	
				$34.22
Every 2 years, painting iron-work			0.50 ÷ .1236 =	4.05
" 7 " floor and laterals—material,		= $4.57		
" 7 " " " —labor,		= 2.50		
		= 7.07 ÷ .5033 =		14.04
" 8 " truss timbers—material,		= 2.08		
" 8 " " " —labor,		= 5.00		
		= 7.08 ÷ .594 =		11.92
Total ultimate cost, per lineal foot				= $64.23

V.—*Combination Bridge as above. Top Chord and End Braces Roofed.*

First cost, preceding case			$34.22	
Roofing			1.50	
Total first cost, per lineal foot			= $35.72	
				$35.72
Every 2 years, painting iron-work			0.50 ÷ .1236 =	4.05
" 7 " timber—material		= $4.57		
" 7 " " —labor		= 2.50		
		= 7.07 ÷ .5033 =		14.04
Every 30 years, timber—material		= 2.08		
" 30 " " —labor		= 5.00		
" 30 " roofing		= 1.50		
		= 8.58 ÷ 4.744 =		1.81
Total ultimate cost, per lineal foot				= $55.62

VI.—*Combination Bridge with Cross Iron Floor Beams, and Wooden Stringers 10′.0 c to c. Intermediate Braces Iron. Top Chord and End Braces Roofed.*

Wrought-iron	414.7 lbs.	@ 5.17c. =	$21.44	
Cast-iron	64.7 "	" 3.88c. =	2.51	
Timber—Y. pine	143′.2 B. M.	" 3.18c. =	4.55	
" W. oak	59′.7 "	" 1.8c. =	1.07	
Framing and raising		=	8.50	
Roofing			1.50	
Total first cost, per lineal foot			$39.57	
				$39.57

Every 2 years, painting iron-work			0.60 ÷ .1236 =	4.85
" 7 " timber—material	= $3.54			
" 7 " " labor	= 1.50 =	5.04 ÷ .5033 =		10.01
" 30 " " material	= 2.08			
" 30 " " labor	= 5.00			
" 30 " roofing	= 1.50 =	8.58 ÷ 4.744 =		1.81
Total ultimate cost, per lineal foot				$56.24

VII.—*Combination Bridge with Longitudinal Iron Floor Beams. Intermediate Braces and Bottom Struts Iron. Top Chord and End Braces Roofed.*

Wrought-iron	476.2 lbs.	@ 5.16c. =	$24.57	
Cast-iron	69.0 "	" 3.88c. =	2.68	
Timber—Y. pine	84'.3 B. M.	" 3.06c. =	2.58	
" W. oak	126'.7 "	" 1.8c. =	2.28	
Framing and raising		=	8.00	
Roofing		=	1.50	
Total first cost, per lineal foot			$41.61	$41.61
Every 2 years, painting iron-work			0.60 ÷ .1236 =	4.85
" 7 " timber—material	= $2.78			
" 7 " " labor	= 1.75			
	= 4.53 ÷ .5033 =			9.00
" 30 " " material	= 2.08			
" 30 " " labor	= 5.00			
" 30 " roofing	= 1.50			
	= 8.58 ÷ 4.474 =			1.81
Total ultimate cost, per lineal foot				$57.27

SUMMARY OF RESULTS.

		First Cost.	Ult. Cost.
I.	Iron bridge with cross iron floor beams and iron stringers	$63.33	$75.92
II.	" " " " " " " " wooden "	57.58	73.72
III.	" " " longitudinal " " " long C ties	59.38	74.54
IV.	Combination bridge with cross wooden floor beams and wooden stringers, not roofed	34.22	64.23
V.	Combination bridge with cross wooden floor beams and wooden stringers, roofed	35.72	55.62
VI.	Combination bridge with cross iron floor beams and wooden stringers, roofed	39.57	56.24
VII.	Combination bridge with longitudinal floor beams and long ties, roofed	41.61	57.27

Conclusion.

In the preceding estimates no favor has designedly been shown one form of truss over another, and the details have been made as nearly alike as the nature of the trusses would admit.

All estimates have been given as much in detail as was consistent with a moderate amount of space; this would not admit of itemized bills of material, but includes strains and sections and data sufficient for reasonably complete strain sheets.

More space has been given to combination bridges than to those of iron and steel, not only because Class A is particularly favorable to this variety, but also from the thorough conviction that engineers in general do not fully appreciate their merits. By many engineers a combination bridge is considered as a make-shift—a temporary affair to be indulged in only by companies having but little money and no credit—and that iron bridges are permanent and ultimately the cheapest.

The estimates giving the comparative economy of iron and combination bridges assume the iron-work to last forever by occasional painting, and the timber to be renewed as often as required. On this basis, comparing estimates I and V, the combination bridge is found to be in first cost but 56 per cent., and in ultimate cost but 73 per cent. of the iron bridge.

As a matter of fact, and as abundantly proven by experience in this country, the iron bridge cannot be considered of much greater permanence than the combination. The latter, with the top chord and end braces roofed, and by the occasional renewal of the floor timber, provided for in the estimates, will last for thirty years, which is as long as the average life of the iron bridge; for some roads in this country are now using the third generation of iron bridges.

The present tendency is to increase and concentrate loads, both propelling and following, and there is good reason to suppose that the experience of the past will be repeated in the future. It is still an unsettled question whether iron does not deteriorate by long-continued use. Engineers are becoming more skilled year by year, and gaining additional knowledge of materials. For these reasons, it would appear to the writer that a bridge which is comparatively cheap, both in first and ultimate cost, which is safe according to present standards, and which can be replaced or strengthened at the least outlay when these

standards change, is the proper one to build when true economy is consulted.

The comparative economy will vary considerably with locality. In the immediate vicinity of the saw-mills the combination plan is still more favorable, while the prices allowed for timber will place it in nearly any part of the United States.

Whenever it becomes necessary to renew the wooden parts, if it should then appear that iron can be substituted at a less ultimate cost, the change can be made with the sacrifice of little except the castings.

Objection is frequently made to combination bridges on the ground that they are liable to bad framing, but good framing can be commanded at the prices named. The framing not less than the iron-work is open to inspection, and as bad work can be found in one as in the other. It is true that combination bridges are more subject to injury from fire than those of iron, as in the latter the flooring only is combustible; nevertheless, losses from this cause are comparatively rare.

If it shall appear from future experiments that the Thilmany method of preserving timber, which renders it also fire-proof, does not injure its strength and consequent fitness for bridge purposes, nothing more can be desired.

Of Class A, the first bridge erected was over Wapsiepinicon River, on the Burlington, Cedar Rapids and Northern Railroad, in 1881; span 147′.4 from c to c of end pins. It is combination, with wooden floor beams (see estimate), essentially the same as shown in detail on Plate XXXI, except that the centre braces are wood. When tested with the weight of three engines its deflection was ¾ inch.

In 1882 some bridges were erected on the Louisville, New Albany and St. Louis Railroad—one of 75 feet through, combination (Fig. 1), and one of 74 feet deck, all iron, over Whiskey Run; also one of 124 feet deck, combination (Fig. 3), over Little Blue River. The above bridges, the writer is informed, are all giving perfect satisfaction.

CLASS A (See Description)

Fig.1

Fig.2

Fig.3

Fig.4

Fig.5

Fig.6

Fig.7

Fig.8

Fig.9

Fig.10

Fig.11

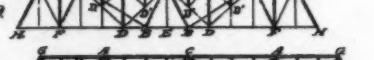
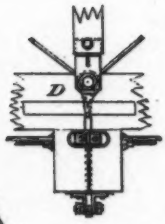


Fig.12

Fig.13

Fig.14



CLASS A (See St

Fig.1

Fig.2

Fig.3

Fig.4

Fig.5

Fig.6

Fig.7

Fig.8

Fig.9

Fig.10

Fig.11

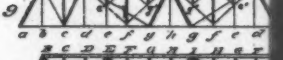


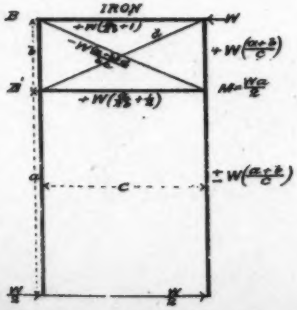
Fig.12

Fig.13

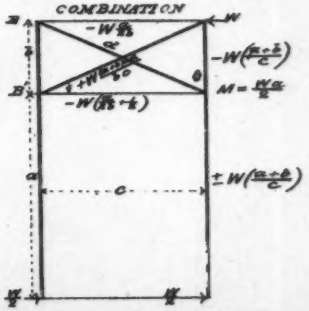
Fig.15

PORTAL BRACING.

IRON



COMBINATION

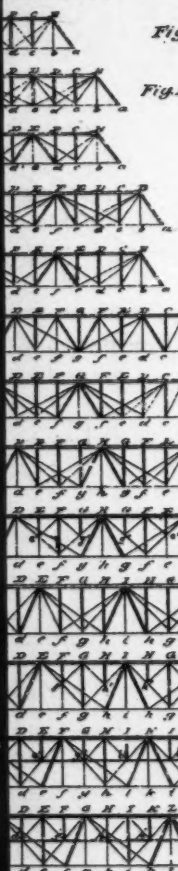


GENERAL

CLASS A (See Strains in Sections)

Fig. 14

Fig. 15



CLASS B

Fig. 1

Fig. 2

Fig. 3

Fig. 4

Fig. 5

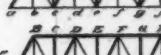


Fig. 1

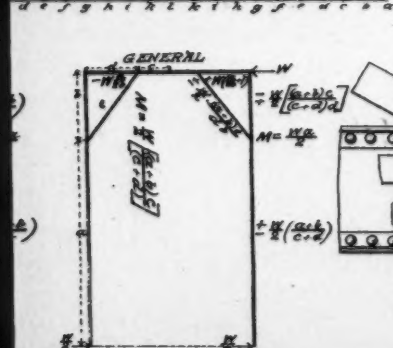
Fig. 2

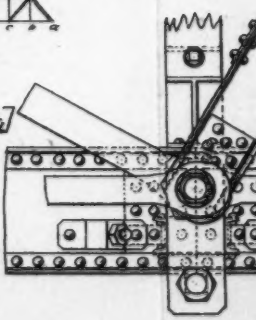
CL

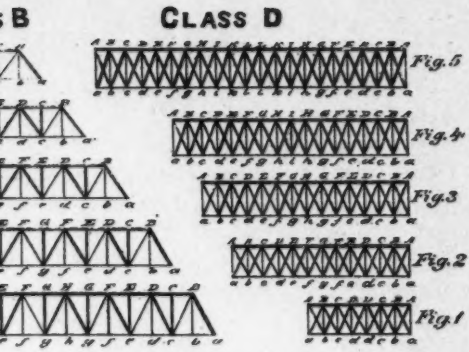
Fig. 1

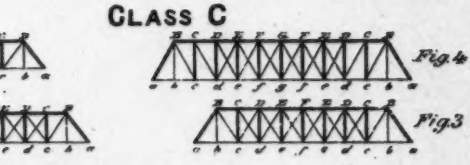
Fig. 2

Fig. 3









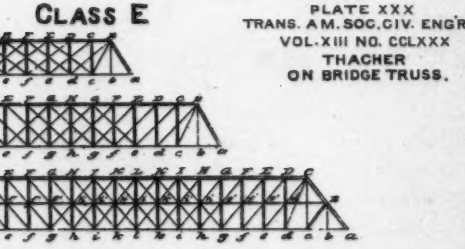
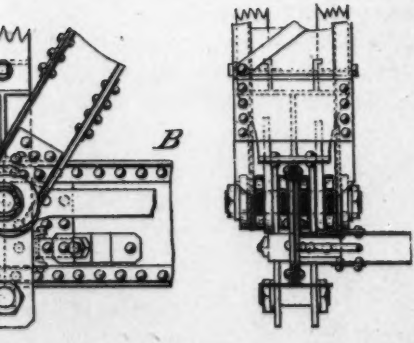
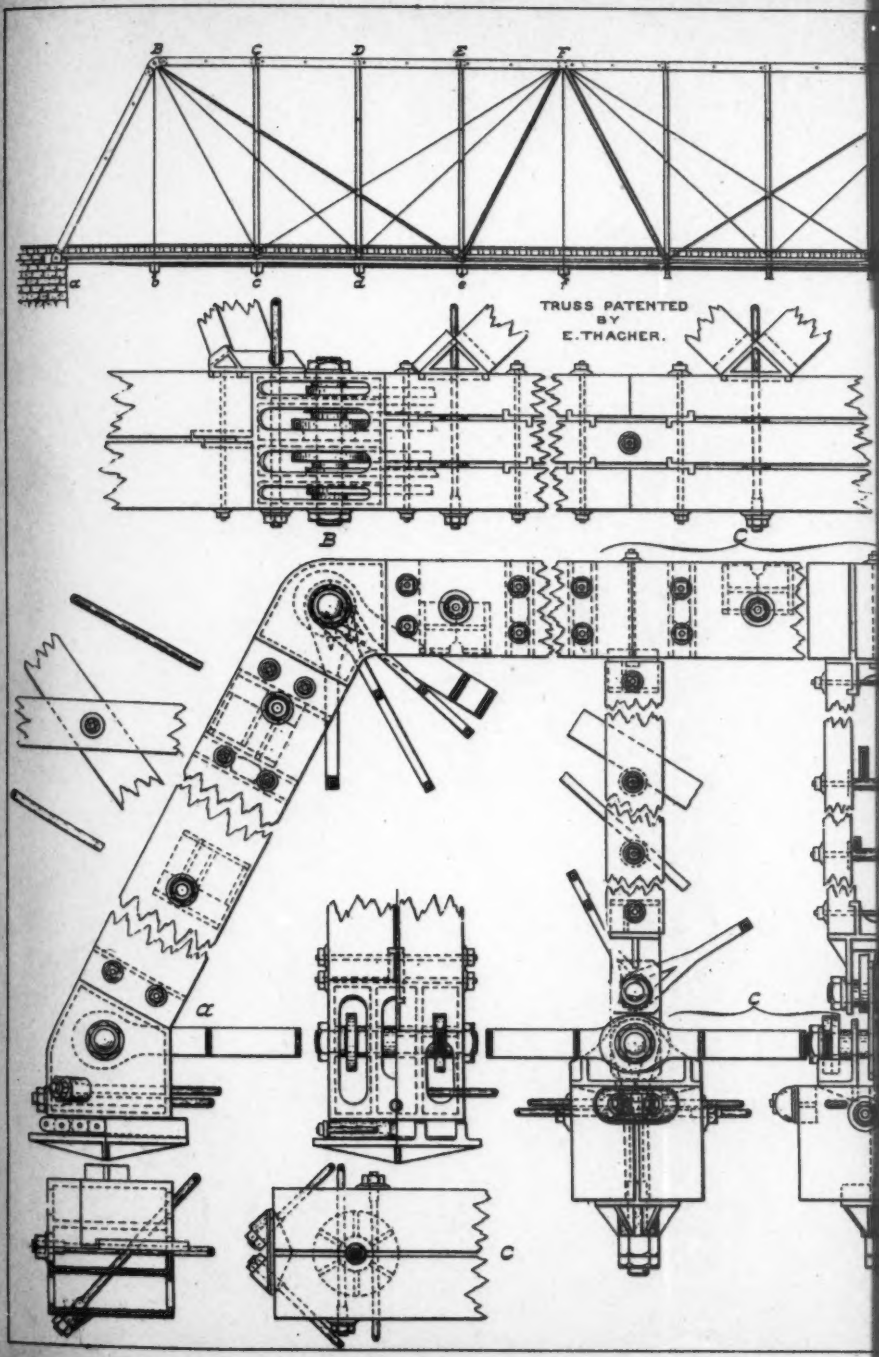


PLATE XXX
TRANS. AM. SOC. CIV. ENGRS
VOL. XIII NO. CCLXXX
THACHER
ON BRIDGE TRUSS.




B
C
D
E
F
a
b
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e
f
TRUSS PATENTED
BY
E. THACHER.
B
C
a
C
C

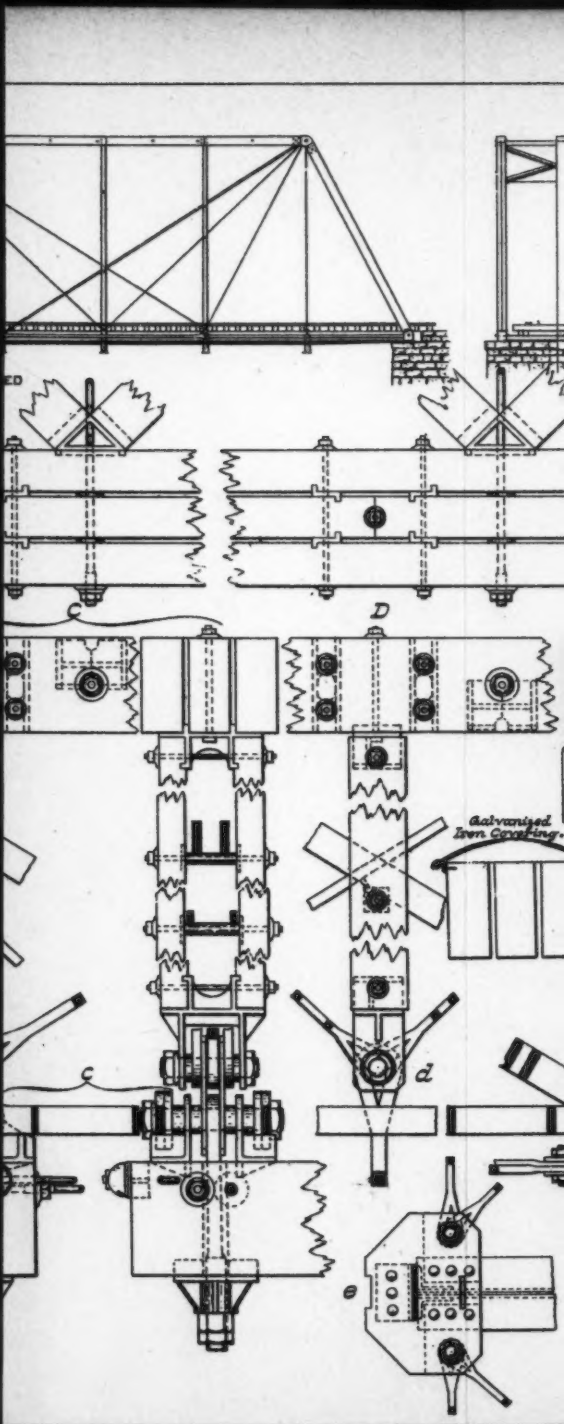
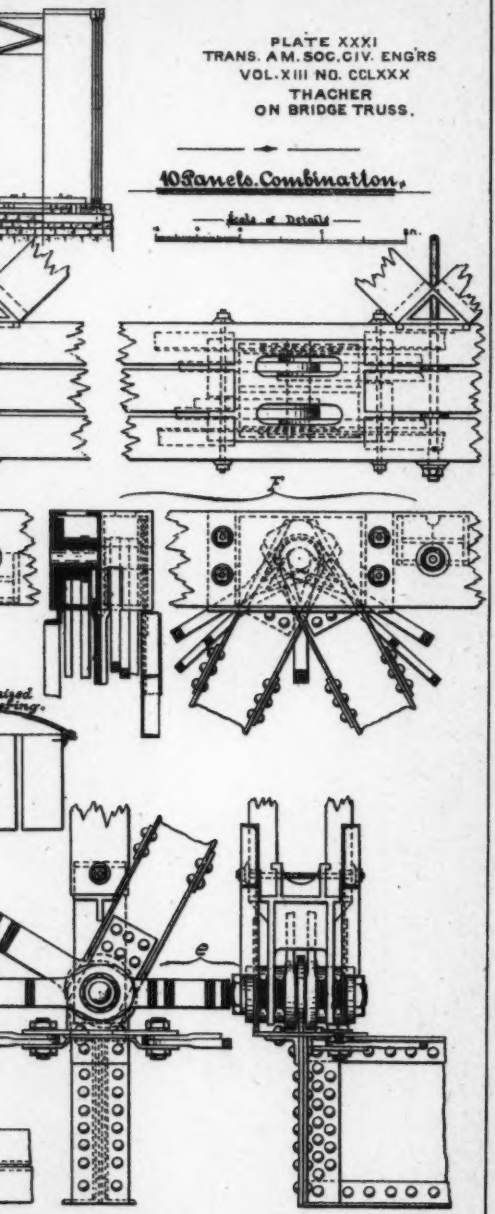

C
D
Galvanized Iron Covering
c
d
e

PLATE XXXI
TRANS. A.M. SOC. CIV. ENG'RS
VOL. XIII NO. CCLXXX
THACHER
ON BRIDGE TRUSS.

10 Panels. Combination.